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State Water Project
SEISMIC LOADING CRITERIA REPORT

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State Water Project
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FOREWARD

The State Water Project (SWP) is considered a critical lifeline for California. Earthquakes pose one of the greatest risks to this vital lifeline. Therefore, the selection of seismic loading criteria becomes critical when designing new facilities or evaluating the safety of existing facilities. This report provides design engineers with a guideline in selecting appropriate seismic loading criteria for a wide variety of SWP facilities including dams, canals, pipelines, tunnels, check structures, bridges, buildings, pumping and power plants, and utility overcrossings. The seismic design load shall be selected based on the criticality of a facility and consequences of failure. Most critical facilities are expected to be functional immediately after an earthquake and thereby should experience very limited damage. Other facilities may be considered less critical such that they are designed to incur some damage but still return to some level of function in a specified timeframe.

These guidelines are a suggested starting point, but do not take the place of the design engineer's judgment and additional information available for a particular project site. Each design engineer should have the knowledge, experience, and insight into the importance of their facility to select the appropriate seismic design load and subsequently to apply that load in an appropriate manner to the structure. Similarly, this report does not prescribe the procedure or process of analyzing the structure. Again, this is design engineer's responsibility to select the method of analyses that best suit the complexity, criticality, and importance of the facility.

This document captures the current state of practice. As the state of practice in earthquake engineering and seismology continually changes, this document shall be reviewed and updated periodically to ensure that SWP facilities are always in accordance with current practice.

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ACRONYMS

A	Acceleration Coefficient
AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
ADT	Average Annual Daily Traffic
AISC	American Institute of Steel Construction
ALA	American Lifelines Alliance
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
AWS	American Welding Society
AWWA	American Water Works Association
CAP	Condition Assessment Program
CBC	California Building Code
CBEA	Consulting Board for Earthquake Analysis
CR	Consequence Rating
CWC	California Water Code
DFM	Division of Flood Management
DOE	Division of Engineering
DS	Design Spectrum
DSHA	Deterministic Seismic Hazard Analysis
DSOD	Division of Safety of Dams
DWR	Department of Water Resources
EERI	Earthquake Engineering Institute
ELF	Equivalent Lateral Force
FEMA	Federal Emergency Management Agency
FERC	Federal Energy Regulatory Commission
FHWA	Federal Highway Administration
GIS	Geographical Information System
GMPE	Ground Motion Prediction Equations
GSSDSB	Guide Specifications for Seismic Design of Steel
IBC	International Building Code
IC	Importance Classification
IO	Immediate Occupancy

ACRONYMS *continued*

LADWP	Los Angeles Department of Water and Power
LRFD	Load Resistance Factor Design
LS	Life Safety
MCE	Maximum Credible Earthquake
MDE	Maximum Design Earthquake
NFPA	National Fire Protection Association
NGA	Next Generation Attenuation
NIBS	National Institute of Building Sciences
O&M	Division of Operation and Maintenance
OBE	Operating Basis Earthquake
PFC	Pipe Function Class
PG&E	Pacific Gas and Electric
PGA	Peak Ground Acceleration
PSHA	Probabilistic Seismic Hazard Analysis
SBA	South Bay Aqueduct
SDC	Seismic Design Categories
SEER	Seismology and Earthquake Engineering Resources Group
SLC	Seismic Loading Criteria
SPC	Seismic Performance Categories
SWC	State Water Contractors
SWP	State Water Project
USACE	U.S. Army Corps of Engineers
USBR	United States Bureau of Reclamation
USGS	United States Geological Survey

EXECUTIVE SUMMARY

Seismic loading criteria are developed for various types of structures in the State Water Project (SWP) to use in future designs and evaluations. The SWP system supplies water for almost two third of Californians and about 750,000 acres of irrigated farmlands. It is critical for the California economy.

Seismic risk is considered one of the greatest contributors of all potential risk categories for the SWP. California is considered one of most seismically vulnerable areas in the world and most of the SWP facilities are located in central and southern California, which are seismically active areas.

The purpose of developing seismic loading criteria was to have an economically feasible SWP system that would provide adequate protection against the loss of life, property damage, and interruption of water delivery during and immediately after a seismic event. Most of the existing SWP facilities have not been evaluated since they were built in the 1960s. The knowledge of ground motion predictions and seismic analytical methodologies has significantly progressed since the 1960s.

As part of the development of the seismic loading criteria, current design standards used by Department of Water Resources (DWR), other similar agencies, and regulatory/code entities were gathered and utilized to select appropriate minimum loading levels for different types of SWP facilities. In addition, the process of criteria selection included consideration of the consequences of failure such as loss of life, property damage, and interruption of water delivery. The consequence of interruption of water delivery was somewhat difficult to quantify because DWR delivers water to the water contractors and they either store it in their storage facilities or distribute to the local agencies. The recommended loading criteria include consideration of consequences of failures in the selection process.

The Division of Engineering (DOE) in DWR should use the seismic loading criteria recommended in this report for the design and evaluation of both new and existing SWP facilities. Seismic loading criteria is provided for most of the structures associated with the SWP—reservoirs, dams, canals, pipelines, tunnels, check structures, pumping and power plants, buildings, bridges and utility overcrossings of the aqueduct. The seismic loading criteria should be used for these structures and associated appurtenant facilities in the future. A project engineer can use a larger loading criteria than what is recommended in this report based on project specific factors such as: 1) the consequences of interruption of water delivery, 2) the impact on existing habitats and the environment, 3) the operational aspects of the facility in relation to water delivery, 4) the repair cost and time to return the system back to operational status, and 5) the availability of an emergency back-up system to temporarily make the facility operational until all the repairs are completed.

The criteria recommended in this report are considered current; however, these seismic loading criteria should be reviewed and updated every five years to make appropriate changes based on future operational requirements and on changes in state-of-practice.

This is the first time DWR has developed loading criteria for the entire SWP facility and many of these facilities have not been evaluated to determine how vulnerable the system would be based on any seismic criteria. Therefore, these criteria are considered preliminary until the recommendations can be confirmed to be reasonable for the SWP facilities.

1.0 INTRODUCTION

Seismic Loading Criteria (SLC) are being developed to provide guidance to design engineers in the California Department of Water Resources (DWR) for determination of the minimum seismic loading requirement for the design or retrofit of State Water Project (SWP) facilities. The seismic loading criteria are necessary for SWP facilities to minimize loss of life, interruption of water supply, and property damage resulting from a seismic event.

These SLC are considered preliminary because the required level of performance of the SWP facilities following a major earthquake was not fully defined when this report was prepared. This is the first time Division of Engineering (DOE) in DWR has developed seismic loading criteria to use internally for the design or evaluation of SWP facilities. The seismic loading criteria considered in the design of SWP facilities in the 1960s were minimal compared to the current standard.

The goal of this report is to provide recommendations on design criteria that are considered appropriate for evaluating existing facilities and designing new facilities. These SLC should be updated periodically to reflect the current state of practice in seismic design and to satisfy growing populations and the demand for water supply. Until the next update is available, DOE design engineers should use this document as a guideline when evaluating existing facilities or designing new facilities.

It should be noted that this report mainly focuses on the seismic loading criteria and does not address in detail the performance requirements or procedures for analyzing the facilities. A facility's performance requirements depend on the project goal, water supply requirements, criticality of the facilities, and many other factors. For example, critical facilities that are expected to be functional immediately after an earthquake should experience very minor to no damage and other facilities that are not critical can experience some damage depending on their functionality, damage consequences, and acceptable repair time and cost. Each type of SWP facility presents a unique set of design challenges. The designer must determine the appropriate methods and level of refinement necessary to design and analyze each type of structure on a case-by-case basis. The designer must also exercise engineering judgment in the application of the seismic loading criteria provided in this report.

When situations arise that warrant detailed attention beyond what is provided in these SLC, the designer should refer to other resources to establish the correct course of action. In 2008, DOE formed an internal committee, the Seismology and Earthquake Engineering Resources Group (SEERG), who can be consulted to discuss the issues and to obtain recommendations. Deviations to these criteria shall be reviewed and approved by the Senior Engineer assigned to a project or by an appropriate member in SEERG, and shall be documented in the project file.

This report is intended for DWR use for SWP facilities. It reflects the current state of practice at DWR. This report contains references specific and unique to DWR and may not be applicable to other public or private parties and agencies.

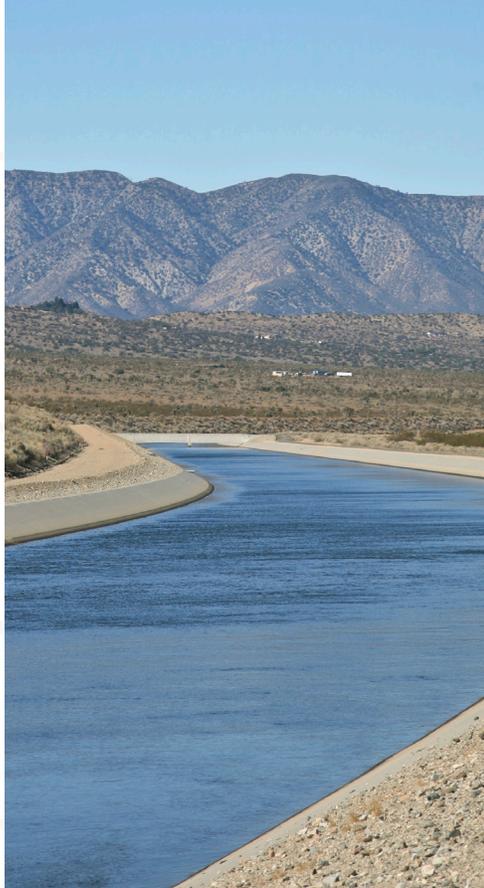
1.1 PURPOSE AND SCOPE

DWR owns and operates water facilities that supply water for millions of Californians and for thousands of acres of agricultural land in California. The SWP was built in the 1960s to convey water from northern California to southern California. These aging facilities require frequent repairs and maintenance to continuously supply water throughout California. Most of the SWP facilities have not been re evaluated since they were built. As part of the Division of Operation and Maintenance's (O&M) "SWP Reliability Study," the SWP facilities will be re evaluated to the current standard to identify deficiencies in the system.

The SWP facilities were built in the 1960s with sound engineering knowledge available at the time of design and construction. However, over the last fifty years, the design and construction standards have improved, especially in the area of seismic design requirements. At the time the SWP was designed and constructed, the understanding of seismic design was limited. Consequently, the design of SWP facilities prompted additional seismic research and development of seismic design criteria. Since then, the understanding of ground motion predictions and analytical methodologies has improved significantly. When comparing the design of these facilities to current standards, seismic loading is considered to be the most vulnerable; potential damage to the SWP facilities could interrupt water supply. A prolonged disruption of water supply for an area that has a sizable population or a large number of industrial infrastructures or farmlands could significantly affect California's economy.

To provide reasonable protection in a seismic event and to maintain consistency in the seismic design of SWP facilities, DOE initiated this project to develop appropriate seismic loading criteria for SWP facilities so that future repairs and designs can be conducted according to the recommendations in this report. As part of developing the seismic loading criteria, the following tasks were conducted:

- Review of seismic criteria used in the design of existing facilities.
- Interaction with other Divisions within DWR and outside agencies to obtain current standard of practice in seismic design of water facilities.
- Development of a Geographical Information System (GIS) database for the entire SWP system.
- Development of preliminary ground motion estimates for various return periods at key locations of the SWP system in order to understand the seismic demand.
- Recommendation of seismic loading criteria for each type of SWP facility.



1.2 PROJECT BACKGROUND

The SWP is a water storage and delivery system comprised of reservoirs, aqueducts, pumping plants, and power plants that extends more than 700 miles across varying California terrain. In July 1956, DWR was created primarily for the construction of the SWP. The first construction efforts began in May 1957 with the Oroville facilities. The construction then continued until the initial SWP facilities (Figure 1.1) (except the peripheral canal) were completed in 1973. Because of funding constraints, construction of the peripheral canal was delayed. While the majority of the SWP facilities were completed in 1973, construction of additional facilities continued to keep abreast of the water delivery obligations to the growing populations in California. Today, the SWP includes 34 storage facilities, reservoirs and lakes, 20 pumping plants, 4 pumping generating plants, 5 hydroelectric power plants, and about 700 miles of open canals and pipelines.

The purpose of the SWP is to divert and store water from the wet season or snow melt and then distribute it to Californians throughout the year. In addition, the SWP is used for recreation, flood-control, power generation, fish and wildlife protection, and water quality management. This unique Project supplies water for 25 million Californians and 750,000 acres of irrigated farmland. The SWP makes deliveries to approximately two-thirds of California's population in 29 urban centers and to agricultural lands in Northern California, the San Francisco Bay Area, the San Joaquin Valley, the Central Coast, and Southern California. The SWP delivers approximately 70 percent of the water to urban users and 30 percent to agricultural users.

The design and construction of the SWP is an ever-changing task because of its unprecedented size and complexity. The seismic component in design was, and remains today, an additional challenge because California is the most seismically vulnerable state in the United States. DWR clearly recognized the importance and need for improved methods of analysis for structure and foundation response to large earthquakes. In the late 1950's, DWR used the seismic analysis provisions included in the Uniform Building Code and the Recommended Lateral Force Requirements of the Structural Engineers' Association of California for buildings and related structures. However, similar levels of analytical procedure or code based methodology were not available for analyzing earth structures and foundations for seismic loads. DWR recognized the need for improving the seismic design procedures and undertook a study that was recommended by the Earthquake Engineering Institute (EERI) in 1960 (DWR Bulletin No. 116-4). In 1961, DWR selected a Consulting Board for Earthquake Analysis to appraise the seismic design problems of the SWP. Since then, various individuals and consulting groups have analyzed seismic issues pertaining to SWP facilities.

DWR has seismic instruments on many of the SWP facilities to monitor real time ground motions. Since the hazard to the SWP facilities from seismic shaking remains high, it is important that SWP facilities are monitored for nearby earthquakes that could cause damage to these facilities. The Earthquake Engineering Section in O&M currently operates 22 sensitive (weak-motion) seismic monitoring instruments at 8 regional locations, along with 103 strong-motion earthquake-monitoring instruments at 42 state-owned facilities for the SWP. These instruments provide timely information about earthquake locations, magnitudes, and severity. When an earthquake of magnitude 3.7 or greater occurs in California, O&M notifies appropriate personnel in DWR, other State agencies, federal agencies and others.

The current design trend in the industry is towards risk-based design for major facilities such as dams and nuclear power plants. Federal agencies, including the United States Bureau of Reclamation (USBR) and the Army Corps of Engineers (USACE), recognize the importance of a risk-based design. They consider the performance of existing structures using multiple levels of seismic loads to evaluate the risk. Based on the results, they identify and repair the most critical structures to restore them to an acceptable risk level. This full risk based analysis is not yet widely used in new designs. In this study, DWR considers risk and the consequences of failure in recommending reasonable seismic loading criteria for various types of SWP facilities.



1.3 REPORT ORGANIZATION

As a part of this study, seismic loading of existing SWP structures was reviewed and compared with current seismic demand. To identify the required seismic demand for various levels of seismic loading, preliminary ground motions (acceleration response spectra) were developed at 18 locations along SWP facilities. These demands were compared to the design values (if available) of the existing facilities to understand the condition of the existing facilities. In order to select appropriate loading criteria, the current design standards used by other agencies as well as the risk and consequences associated with the failure of a facility were considered for each type of SWP facility.

This report includes an individual chapter for each type of SWP facility and provides details regarding the existing facilities, the current standards, if any, used by DWR and other agencies, the recommended minimum seismic loading criteria, and some guidelines on how to select appropriate loading criteria.

Chapter Two presents the criteria and other details about the storage facilities in the SWP system. According to the California Water Code (CWC), most of these facilities fall under DWR's Division of Safety of Dams (DSOD) jurisdiction because of the potential high consequences associated with the failure of a dam or other water retaining structure; only a few existing facilities are non jurisdictional facilities. DOE recommends adopting the existing DSOD criteria for jurisdictional facilities and developed minimum criteria for the non jurisdictional facilities.

Chapter Three presents the seismic loading criteria and the details of facilities associated with the California Aqueduct. These facilities includes canals, pipelines, tunnels, and check structures. The check structures are used to control the flow in the canals. Selecting appropriate seismic loading criteria for canals were difficult because no published information for these types of facilities was found internally or by other agencies that were contacted or researched via the internet. Our recommendation of how to select the loading level for canals and other water conveyance facilities are provided in this chapter.

Chapter Four presents the loading criteria for the pumping plants, power plants, and buildings that are part of the SWP. These criteria are based on the current standards used by DWR and other agencies. The current building code standard is mostly used for these structures with some slight modifications for critical structures as is discussed in Chapter Four.

Chapter Five provides the criteria and details of bridge structures that are part of the SWP facilities. Several bridges owned by DWR cross the California Aqueduct. Generally, Caltrans seismic loading criteria are used for bridge structures.

Chapter Six presents the criteria for utility overcrossings, particularly those that cross canals. The failure of some utility overcrossings can potentially contaminate the water in a canal, interrupting water delivery.

Chapter Seven presents a list of references used in the report.

The initially *estimated* ground motions at the 18 locations along the SWP are presented in Appendix A along with fault rupture hazards. The ground motions provided are **not** to be used for future projects because most of them were developed for pumping plants and used an approximate shear wave velocity in the upper 30 meters based on the limited review of the existing information.

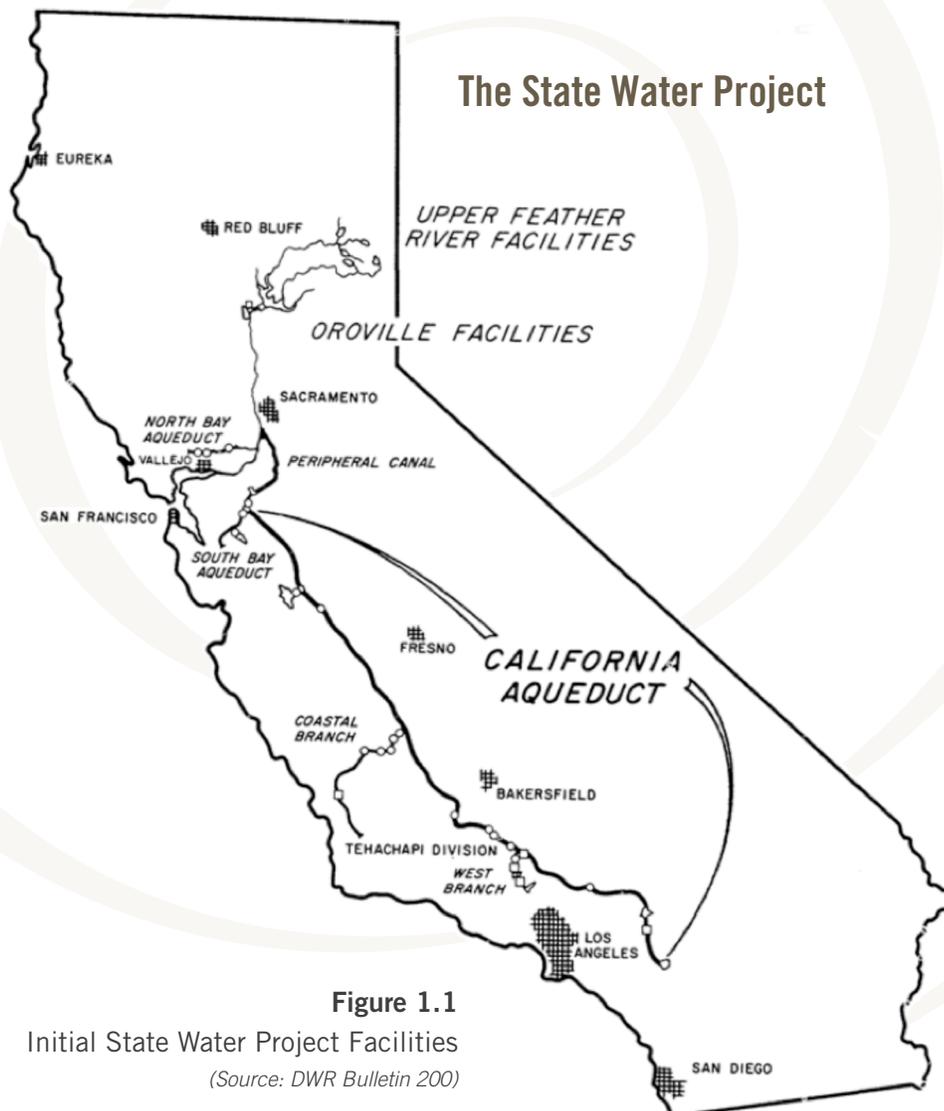


Figure 1.1
Initial State Water Project Facilities
(Source: DWR Bulletin 200)

2.0 STORAGE FACILITIES

DWR owns and operates many storage facilities throughout California, including reservoirs with dams, circular tanks, and small pools or detention basins. This chapter focuses on the SWP storage facilities, the seismic design methodology that DWR utilized to design these features, and seismic loading recommendations for future seismic design and evaluation of DWR storage facilities. This chapter also summarizes the seismic design methodologies of storage facilities utilized by other agencies such as the U.S. Army Corps of Engineers (USACE) and the U.S. Bureau of Reclamation (USBR).

2.1 EXISTING DWR FACILITIES

Currently, there are 29 storage facilities in the SWP. In addition, Citrus Reservoir is under construction. Table 2.1 presents a brief statistical summary of the 29 storage facilities and their dams, as appropriate. The storage facilities range in size from approximately 11 acre-feet (AF) of storage at the Cordelia Pumping Plant Forebay to 3.5 million AF at Lake Oroville. Most SWP storage facilities consist of a reservoir with associated dam or dams. However, the Napa Turnout and Santa Clara Terminal Reservoirs are circular, above ground, steel storage tanks.

DSOD regulates the majority of the dams in the SWP. Based on the CWC, DSOD defines a jurisdictional dam as any dam with a height of 25 feet or greater and a storage capacity of 15 AF or greater, and any dam with a height of 6 feet or greater and a storage capacity of 50 AF or greater. Dams with lesser height and storage combinations than these are considered non-jurisdictional. DSOD also exempts circular storage tanks (e.g. Napa Turnout and Santa Clara Terminal Reservoirs) as well as federally owned storage facilities (e.g. B.F. Sisk Dam) from their jurisdiction.

DWR constructed and currently operates the majority of the SWP storage facilities except for the San Luis Facilities (also known as the Joint-Use Facilities) and Elderberry Forebay. USBR constructed and owns the San Luis Joint-Use facilities; however, DWR operates and maintains it. Similarly, the Los Angeles Department of Water and Power (LADWP) built the Elderberry Forebay; however, DWR maintains and operates it.



Table 2.1 DWR Storage Facilities

Storage Facility	Reservoirs			Dams			
	Gross Capacity ¹ (ac-feet)	Surface Area (acres)	Shoreline (miles)	Structural Height (feet)	Crest Elevation ² (feet)	Crest Length (feet)	Volume (cubic yds)
Frenchman Lake ³	55,477	1,580	21	139	5,607	720	537,000
Antelope Lake ³	22,566	931	15	120	5,025	1,320	380,000
Lake Davis ³	84,371	4,026	32	132	5,785	800	253,000
Lake Oroville ³	3,537,577	15,805	167	770	922	6,920	80,000,000
Thermalito Diversion Pool	13,328	323	10	143	233	1,300	154,000
Fish Barrier Pool	580	52	1	91	181	600	10,500
Thermalito Forebay ³	11,768	630	10	91	231	15,900	1,840,000
Thermalito Afterbay ³	57,041	4,302	26	39	142	42,000	5,020,000
Cordelia Pumping Plant Forebay	11	2	0.3	34	34	1,100	
Napa Turnout Reservoir	22	0.7	n/a	n/a	n/a	n/a	n/a
Clifton Court Forebay ³	28,653	2,109	8	30	14	36,500	2,440,000
Bethany Reservoir ³	4,804	161	6	121	250	3,940	1,400,000
Patterson Reservoir ³	100	4	0.3	33	712		
Lake Del Valle ³	77,106	1,060	16	235	773	880	4,180,000
Santa Clara Terminal	9	0.5	n/a	n/a	n/a	n/a	n/a
San Luis Reservoir (B.F. Sisk Dam)	2,038,771	12,700	65	385	554	18,600	77,645,000
O'Neill Forebay	56,426	2,700	12	88	233	14,350	3,000,000
Los Banos Detention	34,562	623	12	167	384	1,370	2,100,000
Little Panoche Detention	13,236	354	10	152	676	1,440	1,210,000
Tehachapi East Afterbay ³	8006	60					
Silverwood Lake ³	74,970	976	13	249	3,378	2,230	7,600,000
Devil Canyon Afterbay ³	50	4					
Crafton Hills ³ Reservoir	120	6	2.1	95	2932	500	144,000
Lake Perris ³	131,452	2,318	10	128	1,600	11,600	20,000,000
Quail Dam ³	7,580	290	3	45	3,330	6,600	1,900,000
Pyramid Lake ³	171,196	1,297	21	400	2,606	1,090	6,860,000
Elderberry Forebay ³	28,231	460	7	200	1,550	1,990	6,000,000
Castaic Lake ³	323,702	2,235	29	425	1,535	4,900	46,000,000
Dyer Reservoir ³	515	24	0.75	30	810	2,100	150,000

¹ At maximum normal operation level; ² Above mean see level; ³ Jurisdictional Dam

2.2 DESIGN LOADING CRITERIA USED BY DWR AND OTHER AGENCIES

Seismic design of the SWP storage facilities generally consisted of the application of a pseudo-static earthquake load to the critical sliding surface identified in a slope stability analysis. The earthquake loading was derived from a suite of acceleration spectra curves recommended by the 1962 Consulting Board for Earthquake Analysis (CBEA) as the “best current estimates for design of certain structures for a ‘maximum’ earthquake.” The CBEA also provided multiplication factors to increase the recommended curves based upon the site distance to faulting and potential energy release of the fault (Figure 2.1). Typically the loading consisted of a 0.1 g to 0.15 g acceleration. Most of the larger dams in the SWP also implemented additional earthquake design considerations as recommended by the individual project’s Dam Consulting Boards. These additional earthquake design considerations varied from project to project and included model tests using shake tables at higher earthquake accelerations. Incidentally, these tests were some of the earliest tests performed by the Engineering Materials Laboratory at the University of California under the supervision of Professor H. B. Seed. These tests assisted in evaluating design considerations, such as flattening embankment and cut slopes, constructing impervious cores such that they can deform plastically without significant cracking, enlarging impervious cores and transition zones to accommodate potential displacement, and providing additional freeboard.

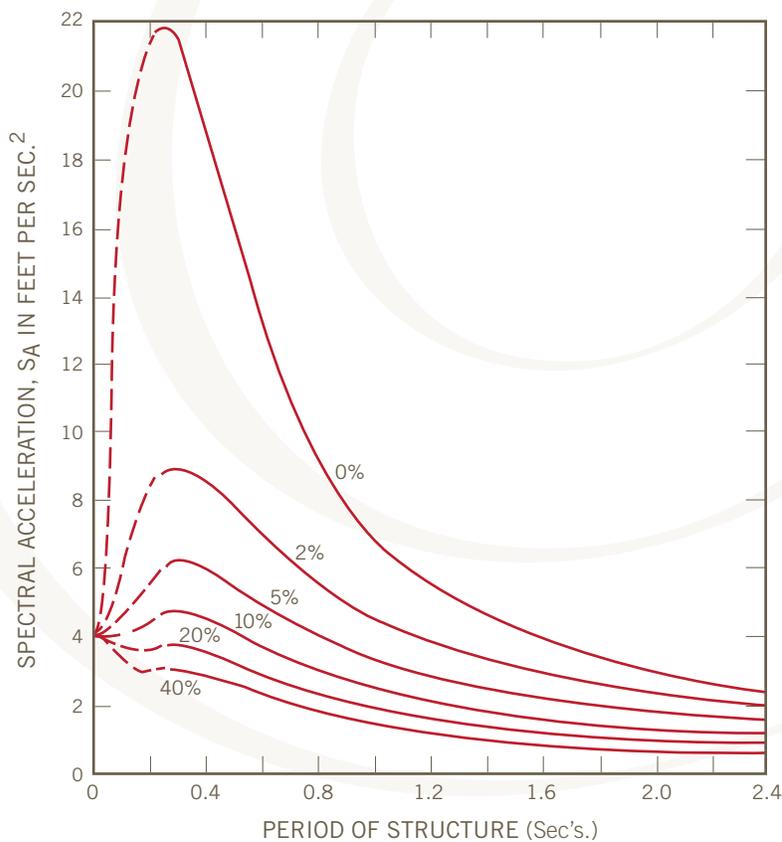


Figure 2.1

Average Acceleration Spectra Curves Proposed by the Consulting Board for Earthquake Analysis in their November 1962 Report

DWR's storage facilities can be grouped into jurisdictional dams and non jurisdictional facilities. As discussed earlier, the non-jurisdictional facilities include circular tanks, federally owned dams, and very low hazard dams. The following sections discuss the current design approaches DWR and other agencies currently use.

2.2.1 Design of Jurisdictional Storage Facilities

As stated previously, DSOD regulates jurisdictional facilities. DSOD also published guidelines for determining earthquake design loading for jurisdictional facilities. Of particular interest to the subject of this report is DSOD's "Guidelines for use of the Consequence-Hazard Matrix and Selection of Ground Motion Parameters" (DSOD, 2002). The Consequence-Hazard Matrix (Table 2.2) prescribes the statistical (deterministic) level Peak Ground Acceleration (PGA) and spectral acceleration based upon Total Class Weight of the facility and slip rate of the controlling fault. The Total Class Weight--a damage potential parameter DSOD uses to evaluate spillway capacity and frequency of facility inspections--is used to represent the range of failure consequences while the slip rate is used as a measure of the likelihood of the controlling earthquake event. The Hazard Matrix requires the use of 84th percentile ground motion parameters for dams with high consequences of failure and/or high slip rate controlling faults and the use of 50th percentile ground motion parameters for dams with lower consequences of failure and/or low slip rates. The guideline also provides procedures to account for near fault directivity effects and establishes minimum earthquake parameters for facilities in areas of low seismicity. Currently, DSOD is considering modifying the hazard matrix shown in Table 2.2. DOE should adopt changes to this hazard matrix as they become available.

		<i>Slip Rate</i>			
		Very High 9 or greater mm/yr	High 8.9 to 1.1 mm/yr	Moderate 1.0 to 0.1 mm/yr	Low less than 0.1 mm/yr
<i>Consequence</i>	Extreme Total Class Weight 31-36	84 th	84 th	84 th	50 th to 84 th
	High Total Class Weight 19-30	84 th	84 th	50 th to 84 th	50 th to 84 th
	Moderate Total Class Weight 7-18	84 th	50 th to 84 th	50 th to 84 th	50 th
	Low Total Class Weight 0-6	50 th	50 th	50 th	50 th

Table 2.2
DSOD Hazard Matrix
(10/4/2002)

USACE has also published guidelines to determine the level of earthquake loading for the design of dams. The Federal Energy Regulatory Commission (FERC), Federal Emergency Management Agency (FEMA), and USBR all refer to USACE guidelines. These guidelines for earthquake loading (ER 1110-2-1806 Earthquake Design and Evaluation for Civil Works Projects) are currently being updated. (The USACE website indicates that the upcoming Engineer Manual 1110-2-6001 Seismic Stability Evaluation of Embankment Dams is currently in the final review stage.) ER 1110-2-1806 establishes combinations of earthquake loading and facility performance. A Maximum Credible Earthquake (MCE) is defined as the greatest earthquake that can reasonably be expected to be generated by a specific fault. Multiple MCE's may be defined for a site, each with characteristic ground motion parameters and spectral shape. The MCE is determined by a Deterministic Seismic Hazard Analysis (DSHA). A Maximum Design Earthquake (MDE) is the maximum level of ground motion for which a structure is designed or evaluated. The MDE can be characterized as a deterministic or probabilistic event. The performance requirement associated with a MDE is that the facility or structure performs without catastrophic failure, although severe damage or economic loss may be tolerated. For critical features, the MDE is the same as the MCE. For other features, the MDE is a lesser earthquake than the MCE which provides economical designs while meeting appropriate safety standards.

The Operating Basis Earthquake (OBE) is an earthquake that can reasonably be expected to occur with a 50 percent probability of exceedance during the service life. The associated performance requirement states that the project functions with little or no damage and without interruption of function. The purpose of the OBE is to protect against economic losses from damage or loss of service, and therefore alternative choices of return period for the OBE may be based on economic considerations. The OBE is determined by a Probabilistic Seismic Hazard Analysis (PSHA). It is apparent from the previous descriptions that USACE guidelines for earthquake loading do not provide a minimum return period or statistical level of ground motion for design other than for the OBE.

Currently, USBR uses a risk-based approach for decision making and to remediate existing dams. In this approach, various failure modes are investigated and the probability of failure for each of the failure modes is calculated. These failure probabilities are then combined with the potential consequences. USBR currently considers only population at risk as a consequence. If the consequence is high, then further action is warranted to bring the consequences below the acceptable level (Cyganiewicz and Smart, 2000).

2.2.2 Design of Non-Jurisdictional Storage Facilities

There are many standards available for the design of liquid-containing storage tanks, three of which would be suitable for the water storage tanks included in the SWP: the American Water Works Association (AWWA) D100, American Concrete Institute (ACI) 350.3, and the International Building Code (IBC) or California Building Code (CBC). The various branches of the United States military also have liquid storage tank design criteria, but these guidelines primarily follow the procedures contained in the previous three codes. Few design standards exist for the design of small reservoirs or dams that DSOD considers non-jurisdictional. The USBR "Design of Small Dams" report does contain useful

information. However, it does not provide seismic design guidance other than to seek the services of an earthquake engineering professional if it is considered possible that an earthquake could affect the facility in question. As stated earlier, USBR currently uses a risk-based approach for analyzing their dams. Currently, many of USACE's military guidelines that could potentially provide seismic loading criteria for smaller facilities and ground-supported tanks are awaiting revision pending the release of Engineer Manual 1110-2-6001, Seismic Stability Evaluation of Embankment Dams.

At this time, information regarding the seismic loading criteria that was used for the design of SWP non-jurisdictional facilities has not been located. It is assumed that if seismic loading was considered for the dams or embankments that impound the smaller reservoirs, the criteria would have consisted of the application of an acceleration factor of approximately 0.1 g for an embankment stability analysis, and possibly flattening of fill or cut slopes. The South Bay Aqueduct (SBA) Terminal Tank (Santa Clara) (construction completed in 1965) and the Napa Turnout (construction completed in 1974), likely had the seismic loading criteria based on the report submitted by the Consulting Board for Earthquake Analysis (CBEA) on November 19, 1962 or by using appropriate codes (if they existed) for that time.



Currently, DWR uses the AWWA and CBC standards for design of steel tanks and the ACI 350.3 for concrete storage tank structures for SWP facilities. Seismic loading criteria employed in the AWWA D100, CBC and ACI 350.3 standards are derived from American Society of Civil Engineers (ASCE) 7-05. ASCE 7-05 was first included in the 2006 IBC (2007 CBC). It was based on an MCE ground motion, which is defined as the motion caused by an event with a two percent probability of exceedance within a 50 year period (recurrence interval of approximately 2,500 years). However, it is limited in regions of higher seismicity to 1.5 times the median estimate of the deterministic ground motion resulting for a characteristic event.

The CBC employs the ASCE 7-05 seismic loading criteria directly while AWWA D100 and ACI 350.3 apply various modifications to ASCE 7-05 to develop the seismic loading criteria. The reader is referred to the various standards for detailed description of the procedures used to determine the seismic loading criteria for steel and concrete storage tanks.

2.3 RECOMMENDED SEISMIC CRITERIA FOR DWR FACILITIES

The recommended minimum seismic loading criteria for SWP storage facilities are based on a deterministic or probabilistic spectrum depending upon whether the facility is jurisdictional or non-jurisdictional. The determination of this design spectra and the application of appropriate adjustment factors are provided below.

2.3.1 Jurisdictional Facilities

The loading criteria for jurisdictional facilities are determined using the DSOD criteria as follows:

The statistical level of ground motion for design (50th- or 84th-percentile) is determined from the DSOD Hazard Matrix (Table 2.2) based upon the consequence of failure (Total Class Weight obtained from DSOD) and the slip rate of the causative fault (obtained from a Seismic Hazard Assessment).

The 50th or 84th percentile deterministic design ground motion is then calculated by taking the average of the Next Generation Attenuation (NGA) Ground Motion Prediction Equations (GMPE) as appropriate. For sites with V_{s30} (average shear wave velocity in the upper 30 meters) less than 450 meters per second, or where ground motions are controlled by dip-slip faulting, the Idriss (2008) GMPE should not be included in the average response.

The design spectrum should be modified to account for fault rupture directivity using the model by Somerville et al. (1997) as modified by Abrahamson (2000) to develop the fault average component. Values of percent rupture towards the site required in the Somerville modifications shall be taken as 40 percent for strike slip faults and 85 percent for dip slip faults.

The design spectrum should be the same or above the minimum earthquake defined by DSOD (DWR, 2002).

Measurements of V_{s30} using in-situ (subsurface) geophysical methods (PS Suspension Logging, Down-hole Seismic, Seismic CPT cone, etc.) where feasible, are preferred specifically for estimating ground motions. Where subsurface methods are not practical, surface geophysical methods (or Rayleigh Wave Inversion – SASW/ReMi) are acceptable. In the absence of geophysical measurements or if limited geophysical measurements are available, V_{s30} for soil and rock can be estimated based upon available subsurface information and/or from using established correlations. For soils and very soft rocks the V_{s30} can be estimated based upon laboratory measured undrained shear strength, SPT blow count value, N_{60} (blow count corrected for hammer efficiency but not for overburden), or the CPT tip resistance. Similarly, for firm to hard rock, V_{s30} can be estimated based on the weathering, type, and quality. An experienced geologist or geotechnical professional should make the V_{s30} estimates.

2.3.2 Non-Jurisdictional Facilities

Non-jurisdictional facilities can be circular storage tanks, large reservoir dams owned by Federal Government (e.g. USBR), or small reservoir dams that do not meet the CWC or DSOD jurisdictional requirements.

2.3.2.1 Circular Storage Tanks

AWWA D100, Section 13, Seismic Design of Water Storage Tanks determines the loading criterion for circular steel storage tanks. ACI 350.3, Chapter Four, Earthquake Design Loads determines the loading criterion for circular concrete storage tanks. The design earthquake ground motion in these standards is derived from ASCE 7-05.

Both standards have a general and site-specific method of determining design response spectra. The general methods are based on an MCE. The site-specific methods define the response spectra as the lesser of a probabilistic response spectrum with a two percent probability of exceedance in a 50-year period and the deterministic spectral acceleration taken as 150 percent of the median response spectra (or the 84th percentile deterministic spectral response acceleration).

The designer should first consult with an engineer who has experience in developing design ground motions to seek advice whether site-specific design ground motions are warranted for a project. For certain subsurface conditions, ASCE 7.05 recommends specific ground motions.

2.3.2.2 Large reservoir dams owned by Federal government

The State does not have jurisdiction over federally owned storage facilities. For example, USBR owns the San Luis facilities and the State of California does not regulate them. As these facilities are operated and maintained by the State, the seismic loading criteria for these facilities should be determined as if they were under DSOD jurisdiction, following the DSOD criteria outlined above in Section 2.3.1.

2.3.2.3 Small Storage Facilities

The non-jurisdictional and non-circular tank facilities (e.g. small Forebays and Afterbays, detention ponds) should have low hazards compared to jurisdictional dams. However, a minimum design criterion is needed to minimize frequent repairs, interruption of water supplies, and other impacts on the public and economy. The minimum loading criteria for these facilities is the envelope of ground motion with a 500 year return period determined from a probabilistic seismic hazard analysis and the median earthquake event from the nearest controlling fault. If the repair cost and impact to the water delivery is significant, the designer could potentially adopt the procedure that is recommended for the jurisdictional facilities.

3.0 CALIFORNIA AQUEDUCT FACILITIES

3.1 BACKGROUND INFORMATION

3.1.1 Existing DWR Facilities

The water conveyance (aqueduct) facility in the SWP consists of a main stem, also known as the California Aqueduct, and five branches--North Bay Aqueduct, South Bay Aqueduct, Coastal Branch, East Branch, and West Branch. The aqueduct facilities are composed of approximately 700 miles of canals and pipelines that have the capacity to hold approximately 118,000 acre-feet of water at any given time. The construction of the initial facilities was completed during the early 1970s when the SWP became operational. Currently, 71 check structures are located within the aqueduct system: 61 in the California Aqueduct, 7 in the South Bay Aqueduct, and 3 in the Coastal Aqueduct. Most of the North Bay Aqueduct and East and West branches consist of pipelines. Check structures were designed to regulate the flow of the aqueduct using multiple radial gates and to isolate the canal into pools. Check structures also provide a vehicle overcrossing of the canal. The pipelines have control valves to regulate the flow.

The seismic hazard on the water conveyance facilities has been a concern for DWR in recent years. Seismic hazards primarily include ground ruptures and ground shaking, along with the secondary effects of these hazards such as liquefaction, landslides, water surge or waves, and ground settlement. The possibility of fault rupture hazards on the SWP facilities are slightly less than the ground shaking hazard from an earthquake. However, the canal crosses active faults at many locations and the impact of potential fault rupture displacements should be considered when analyzing for seismic hazards. Appendix A (Initial Seismic Hazard Determination of SWP Facilities) of this report provides preliminary estimates of ground motions at 18 pre-selected locations along the SWP.

3.1.2 Impact of Failure and Consequences

The failure of major water delivery channels and pipelines may lead to various consequences such as: (1) Heavy economic loss; (2) Mass reduction or termination of potable water; (3) Agricultural, industrial, and fire suppression/emergency response vulnerability; and (4) Severe environmental impacts.

The SWP aqueducts transport water mostly from northern California to central and southern California. DWR delivers water to the State Water Contractors (SWC) to be used by the local cities and water districts. The main goal of the SWP aqueduct system is to provide the contracted water supply to the SWC. A canal or pipeline failure during an earthquake would not only negate water delivery but could



also flood the regions adjacent to the failure. Therefore, the criticality of the systems depends on the operational and flooding consequences. The operational consequences could be significant if the water supply is interrupted for a long period of time and if the local users cannot survive without SWP water for that length of time. Generally, locals have their own storage facilities to continue water delivery to their population for a short period of time in the event of a SWP shut-down. The flooding consequences can also be significant depending on the following parameters: (1) size/length of canal pool or pipe section that failed; (2) location and alignment of the aqueduct relative to the urban or industrial areas; (3) volume of discharge through the canal pool or pipe; and (4) economic, social, and environmental impacts of the failure. Regions where population is high or located in close proximity to a canal, should apply more stringent seismic loading criteria to reduce the economic and life safety impacts.

3.2 SEISMIC LOADING CRITERIA USED BY DWR AND OTHER AGENCIES

3.2.1 Canals

While information regarding the construction of the SWP canals can be located in the Bulletin 200 document and various other design and construction reports, there appears to be very little documentation of the seismic loading criteria that was used for the canals. Geology, soils, and seismicity have been discussed in various reports, but information about how the canal design accounted for seismic activity was only found for the North San Joaquin Division and Coastal Branch. A seismic loading of 0.1g in the horizontal direction was used in both areas during slope stability analyses. With this seismic load, minimum factors of safety, 1.0 and 1.20, were used for both construction and operation conditions, respectively. In the 1960s, liquefaction analysis was not in practice and probably ignored in the design of canals. Since there has been no new canal designs completed in the last several decades, DWR does not have current standards for canal seismic design.



In the attempt to research information in relation to the canal design, DWR engineers that were involved in the design and construction of the SWP were contacted. Based on their experience, it appears that seismicity did not play a significant role during the design of the SWP canals because it was thought that canals could be repaired in a relatively short amount of time if damage were to occur during a seismic event. In addition to the research within DWR, staff contacted the USBR, Pacific Gas and Electric (PG&E), and the LADWP to determine if they had any documentation regarding the seismic loading criteria they used to design their own canals. Unfortunately, no seismic loading criterion for canals was located.



In recent years, DWR has performed levee evaluations for most of the levee systems in the Central Valley. As part of the Urban Levee Geotechnical Evaluations Program within Division of Flood Management (DFM), seismic vulnerabilities of urban levee systems are analyzed using 200-year ground motions as documented in the Draft Guidance Document for Geotechnical Analyses – Version 11, December 2011. Levee systems that protect communities of more than 10,000 people are considered urban levees and are included in this program's evaluation.

3.2.2 Pipelines

Similar to SWP canals, little documentation exists regarding the seismic loading criteria used in the design of existing pipelines including the recently designed pipelines. DWR does not currently use any analytical model to predict the behavior of buried pipelines during earth-

quake occurrences. This is partly because earthquake loads may not be a concern for pipelines below the ground surface. Furthermore, AWWA manuals do not explicitly include seismic loading criteria for water pipelines. FEMA's recommendations are provided below.

3.2.2.1 Federal Emergency Management Agency (FEMA) - Pipelines

Recommendations for the seismic loading criteria for water pipelines can be found in a guideline prepared for FEMA's and National Institute of Building Sciences (NIBS) by a team representing practicing engineers in the United States' water utility industry and academics through American Lifelines Alliance (ALA).

The primary earthquake hazards concerning water pipelines can be classified mainly as transient and permanent ground movements.

1. *Transient Ground Movement*

Transient ground movement describes the shaking hazard by waves propagating from the energy source and the amplifications because of surface and near surface ground conditions and topography. Transient ground movements by seismic waves cause compressive, tensile, and bending strains in buried pipelines by moving pipes with the soil in the area without ground failures. Assuming the strain is transferred to the pipe without slip and the strain on the pipe is equal to the strain in the soil that can be computed by considering the peak ground velocity and wave propagation speed. The maximum force the soil can transfer to the pipe can be estimated from the frictional force of soil acting on a pipe barrel in the axial direction (force per unit length) and the seismic wavelength in the soil at the pipe location.

2. Permanent Ground Movement

The strains on buried pipes because of permanent ground movement are caused by surface fault ruptures, slope movements and landslides, liquefaction-induced lateral spreading and flow failure, and differential settlements. Permanent ground movement caused by an earthquake should be considered for seismic design of pipelines. The amount of surface displacement because of fault offset can be estimated by using models provided by Wells and Coppersmith (1994). Liquefaction induced permanent ground displacement can be estimated by using a model by Bardet et al. (2002) and other recent publications. The average landslide induced permanent ground displacement can be estimated by a model provided by Jibson (1994). Permanent ground movements on buried pipelines have greater impacts than the transient strains from wave passage.

Table 3.1 below summarizes the transient and permanent ground movement hazards that are considered and earthquake parameters needed for an engineering evaluation according to FEMA. The recommended methods for obtaining the parameters are also included.

Table 3.1 Earthquake Hazard and Parameters for Pipeline Design (Source: FEMA 2005)

Hazard	Earthquake Parameters	Obtain from:	Geotechnical Parameters
<i>Transient Ground Movement</i>			
General Shaking	PGA (Peak Ground Acceleration), PGV (Peak Ground Velocity), spectral response	PSHA (Probabilistic Seismic Hazard Analysis)	Soil/rock conditions, depth, Shear wave velocity (V_s)
Near-source Directivity	Fault Distance	PSHA, fault map	Fault type, orientation, rupture direction
Ground Amplification	PGA, PGV, spectral response	PSHA	Site soil and rock conditions, V_s
<i>Permanent Ground Movement</i>			
Faulting	Magnitude, Length	PSHA or Geologist	Fault type, orientation
Liquefaction	PGA, Magnitude	PSHA	Soil type, relative density, thickness, groundwater
Lateral spread and Flow failure	PGA, Magnitude, Distance	PSHA	Topography, soil type, strength, thickness, groundwater
Slope Movement, landslide	PGA, Acceleration time history	PSHA	Topography, ground strength, groundwater
Settlement	PGA	PSHA	Soil type, strength, thickness, groundwater

Table 3.2 below provides the range of pipe function classes based on seismic importance along with a description of the type of pipe. As the purpose of water use still remains descriptive, engineering judgment should be exercised in classifying a pipeline for design decisions. Based on a 50 year design period of pipeline, the earthquake hazard return periods for each pipe function class is also included in Table 3.2 based on the pipe function class.

Table 3.2 Earthquake Hazard Return Period Based on Pipe Function Class

(Source: FEMA, NIBS, American Lifelines Alliance Inc., 2005)

Pipe Function Class	Seismic Importance	Description	Probability of Exceedance, in 50 years	Return Period (years)
I	Very low to None	Pipelines that represent very low hazard to human life in the event of the failure, longer restoration period (2 weeks or longer will not hurt economic well being of community)	100%	Undefined
II	Ordinary, Normal	Normal and ordinary pipeline use	10%	475
III	Critical	Critical pipelines serving large numbers of customers and present significant economic impact to the community or a substantial hazard to human life and property in the event of failure.	5%	975
IV	Essential	Essential pipelines required to remain functional and operation during and following a design earth quake.	2%	2475

3.2.3 Tunnels

The seismic loading criteria that were used in the design of existing SWP tunnels also have not been found. Many references, including the “Seismic Design of Tunnels – A Simple State-of-the-Art Design Approach” monograph (Jaw-Nan Wang and Parson Brinckerhoff, 1993) discuss the seismic loading criteria that could be used for tunnels.

3.2.4 Check Structures

DWR design reports containing seismic loading criteria information for existing check structures were not located. DWR Bulletin 200 does cite that check structures located in the San Joaquin Field Division were designed for seismic loading of 0.1 g, but does not elaborate on the methodology



used to arrive at that particular loading. A recent check structure design for the South Bay Aqueduct (SBA) enlargement utilized the CBC and ASCE 7-05 Minimum Design Loads for Buildings and Other Structures for structural design including seismic loading.

USACE seismic criterion (Earthquake Design and Evaluation for Civil Works Projects, ER 1110-2-1806), USBR design criteria (Water Conveyance Facilities, Fish Facilities, Roads and Bridges – Design Standards No. 3, Draft 2), and ASCE 7-05 should be considered when establishing a seismic loading criteria for these SWP facilities. The following section provides a summary of seismic criteria that are currently being used by the USACE and USBR agencies, and adopted by ASCE.

3.2.4.1 US Army Corps of Engineers (USACE) – Check Structures

The USACE seismic criteria apply to a wide range of structures and should take into consideration the consequences of project failure. USACE uses two types of analysis to select ground motions. The first method uses DSHA which incorporates magnitude, site conditions, and attenuation relationships to select ground motions. The alternative approach is PSHA, which applies a similar process as DSHA, but includes probability of exceedance and structure service life to calculate return period to determine ground motions. The largest earthquake that can be expected based on geological and seismological evidence is the MCE and is determined by DSHA. The MDE is the ground motion to which the structure will be designed and allowed to respond inelastically without collapse. MDE's are characterized as a deterministic or probabilistic event. To protect against loss of service, an OBE is used which has a return period of 144 years (probability of exceedance of 50 percent and service life of 100 years). Structures designed for OBE should respond elastically. As stated previously, the USACE does not provide specific seismic loading criteria for check structures, but does recommend that critical or essential structures should be designed for both a mean and 84th percentile MCE event.

3.2.4.2 US Bureau of Reclamation (USBR)

Based on the USBR's design criteria (USBR Design Standards No. 3, Water Conveyance Facilities, Fish Facilities and Roads and Bridges, Draft – Phase 2 [Reclamation – wide review], September 2009), a seismic event with a ten percent probability of being exceeded in 50 years (475 year return period) is applied if ACI 350 is used for the structural design. Response accelerations are obtained from the United States Geological Survey (USGS) hazard maps. Site specific studies may be required if the structure is located near known faults or soil layering exists which could increase

accelerations more than typical values based on soil profiles. Soil classifications can be found in ASCE 7-05. While USBR does not reference the use of ACSE 07-05 for its seismic loading criteria, it appears their intent is to use ASCE 7-05 for soil classification.

3.2.4.3 American Society of Civil Engineers (ASCE) 07-05

Below is an outline describing the methodology used to determine the design acceleration for a check structure using ASCE 7-05 Seismic Design Criteria (Chapter 11). Based on the structural configuration and performance of check structures, it is expected that design accelerations would be determined using a short period response. Some content from the design criteria that would not be applicable to check structures has been removed for clarity.

1. Seismic Hazard Map
 - S_s – Spectral response accelerations for short period (0.2s)
 - 2475-year return period
 - Five percent critical damping
 - Site class B
 - Probabilistic values with a deterministic cap (in California)
2. Site Class based on soil profile
 - Choice of site class based on soil stiffness (measured differently depending on soil type)
 - Site class A (hard rock) thru site class F (very soft soils)
 - Site class D (stiff soil) is default without sufficient geotechnical data
3. Site Coefficients based on site class
 - Calculate F_a based on site class and response accelerations
 - F_a - Site coefficient at 0.2s
4. MCE Spectral Response Acceleration Parameters
 - Calculate SMS based on site coefficients
 - $SMS = F_a \cdot S_s$ (Equation 11.4-1)
5. Design Spectral Acceleration Parameters
 - Approximately equal to 500 year return period
 - Calculate SDS by reducing MCE acceleration parameters
 - $SDS = \frac{2}{3} \cdot SMS$ (Equation 11.4-3)
6. Design Response Spectrum (if required)
 - Equations in place of site specific ground motion used to generate curve
7. MCE Response Spectrum (if required)
 - Design Response Spectrum multiplied by 1.5

3.3 RECOMMENDED SEISMIC CRITERIA FOR SWP AQUEDUCT FACILITIES

3.3.1 Canals

Two viable options were considered in establishing the seismic loading criteria for existing and future SWP canals. The first option would be to set a standard seismic event or load to be applied to all canals throughout the SWP, irrespective of failure consequences. The second option would be to establish different loading criteria with corresponding consequences of canal failure in various regions along the SWP.

The option of setting a seismic loading standard applicable for the entire SWP's canals should also be considered. A 200-year seismic event is reasonable criteria to evaluate existing and design new canals. As described previously, DWR's Urban Levee Evaluation Program uses a 200-year return period seismic loading to evaluate levees in urban areas. The DWR's Urban Levee Design Criteria report, dated May 2012, recommends using higher than a 200-yr loading for frequently-loaded urban levees. Most of the levee systems primarily carry water during the winter seasons and not year round. However, SWP's canal system carries water throughout the year and thus has a continuous loading. Unlike many levees, canals were constructed with engineered fill and are expected to withstand larger shaking than levees, which generally were constructed without rigorous compaction standards. It should also be noted that since the existing SWP canals were designed for low seismic load (0.1g), establishing a loading criteria that is too high would result in most, if not all, of the SWP canal embankments requiring repair.

Establishing different loading criteria based on the failure consequences is beneficial and potentially cost effective. For areas where high consequences are anticipated, more stringent seismic loading criteria are justifiable. Utilizing a consequence chart will allow canals throughout the SWP to be designed for different loadings depending on various critical factors such as affected population, economic loss, and degree of dependency of local users on the SWP for water supply. Based on these factors, a consequence chart would dictate the recommended level of loading for analysis and/or design of each canal region. Development of the consequence chart is further discussed below.



3.3.1.1 Consequence Chart format for Canals

The Consequence Rating (CR) for each canal pool can be characterized based on the level of hazard in the inundation area. The CR level is based on the estimated population of the inundation area. This assessment assumes that the resulting economic impact to the area is somewhat related to the population at risk in the case of a canal pool failure by a seismic event.

A “Low to Medium Hazard” rating typically means that there is a life safety threat in the inundation area with populations less than 10,000 people. A “Medium to High Hazard” rating means that the inundation area has a larger population of equal to or greater than 10,000 people, and a seismic event would be a large threat to this urban area.

Table 3.3 below describes two CR's, for simplicity, and the associated level of loading that should be assigned to each rating.

Table 3.3 Canal Consequence Chart - CR and Level of Assessment

Consequence Rating	Population	Level of Assessment
Low to Medium Hazard	< 10,000	200 year
Medium to High Hazard	≥ 10,000	500 year

Table 3.4 below provides the canal pools where the CR could possibly be considered a “Medium to High Hazard.” These canal pools should be evaluated for a 500-year ground motion. The entire SWP canal system was evaluated to estimate population for areas within the inundation area. It was assumed that everyone within a three to five mile radius of the canal will be affected by a canal failure given the elevation of the population is below the elevation of the canal. In addition, to estimate the population, it was assumed that every four acres housed one person in areas of medium to high density populated areas. This population to area ratio was estimated by considering the inaccuracy in determining the exact area of the populated regions as well as the presence of open land in developed areas such as parks and schools. The US Census data of 2000 was also used to estimate the population of the larger cities. The 2009 National Agriculture Imagery Program coupled with Google Earth (for elevation determination) was used to determine the areas where populated areas were in the inundation area of each canal pool (see Figure 3.1). For inundation areas where the imagery clearly showed a rural region (which has isolated dwellings), the population was estimated based on the number of dwellings that were counted in the area and estimating four persons per dwelling. An area where the population is 10,000 or more was considered to be an urban area and thus a Medium to High Hazard rating. The level of seismic loading for these areas is recommended to be a 500 year return period. All canal pools that are not listed in Table 3.4 should use a minimum of a 200 year return period.

Table 3.4 Population Analysis Results for Canal Pools based upon 2000 US Census data

Pool	Pool Capacity (ac-ft)	Pool Length (ft)	Estimated Population Potentially Affected by Failure	Level of Assessment
2	1,874	32,030	65,940	500 yr
3	1,939	33,170	79,235	500 yr
7	1,777	29,950	19,033	500 yr
8	1,896	32,000	19,033	500 yr
9	1,667	28,120	10,182	500 yr
10	1,753	29,400	15,280	500 yr
13	11,086	83,840	35,119	500 yr
49	488	26,990	299,651	500 yr
50	537	29,520	299,651	500 yr
51	55	2,960	299,651	500 yr
52	182	8,780	299,651	500 yr
53	480	23,450	299,651	500 yr
54	218	10,920	299,651	500 yr
55	260	13,020	299,651	500 yr
56	182	10,860	299,651	500 yr
57	206	11,360	12,591	500 yr
58	345	19,360	15,026	500 yr
64	470	29,660	88,715	500 yr
65	439	27,580	88,715	500 yr
66	205	16,290	88,715	500 yr



Figure 3.1
Population Analysis
Map in GIS

Figures 3.1 and 3.2 are visual representations of the recommended loading criteria for canals as shown in ArcGIS. Based upon 2000 US Census data, the blue line work represents canal reaches that are recommended to have a loading criterion of a 200 year return period while the red line work indicates a 500-year return period recommendation.

Figure 3.2 Summary of DWR Loading Criteria Recommendations for Canals



**Blue line work – 200-year return period *Red line work – 500-year return period*

3.3.2 Pipelines

The seismic loading criteria for pipelines are mainly based on the recommendations in the guidelines prepared by the ALA Inc (2005) for the FEMA and NIBS, CBC 2007, and ASCE 7 manuals.

To compute the permanent and transient ground movements for SWP pipeline locations, peak ground acceleration, location and orientation of nearby faults, and distance to the earthquake location are needed. These values can be obtained by performing site-specific seismic hazard analysis.

There are two main approaches for computing site-specific ground motions, namely deterministic and probabilistic. Deterministic seismic hazard analyses are commonly used where the variation of the magnitude over an area is less and when examining the performance of a complete pipeline network over a large area is required (FEMA, 2005). The probabilistic approach is based on the probability of occurrences. Probabilistic site specific hazard analysis results can be obtained from USGS hazard maps. The seismic loading criteria for pipelines adopts the probabilistic methodology as a preferred approach; however, it is recommended that the designer consider both probabilistic and deterministic seismic analyses methods in the decision making process.

The Pipe Function Class (PFC) for each reach of pipeline can be characterized based on the seismic importance of the pipeline. It is recommended that the PFC be based on FEMA's recommendations as shown in Table 3.2.

Detailed assessments of each specific pipeline should be undertaken to assign an appropriate function class for retrofitting or new designs. Detailed assessments shall include, but are not limited to, the seismic characteristics of the area, soil properties, economic impacts, environmental impacts, number of people served, and purpose of the pipeline. As a general guide, it is recommended that a 500-year or greater return period ground motion be used for designing SWP pipelines. Most of the SWP pipelines are critical because they serve large populations.

Table 3.2 and a PSHA should be used as a guide to determine PGA for pipeline design. The design earthquake for pipe appurtenances (e.g. surge tank) should match that of the associated pipeline.

3.3.3 Tunnels

The seismic loading criteria considered for the evaluation and future design of tunnels should, at a minimum, follow the loading criteria for pipelines. However, if the potential economic damage or cost of repair for a tunnel facility is high, the recommended criteria shall be increased to a 1,000-year return period or more.

3.3.4 Check Structures

Based on operational protocol, check structures should remain in operation during an emergency. Thus, the minimum design loading for a given check structure should be greater than the criteria for the adjacent canals/pools. Recommendations for the DWR seismic loading criteria regarding check structures should be based on the CBC and ASCE 7-05 Minimum Design Loads for Buildings and Other Structures. Both ACI 318 08 (1.1.9) and ACI 350-08 (1.1.8.2) design code provisions for earth-

quake resistance refer to ASCE 7-05 to determine level of seismic risk. As noted in the codes above, the seismic hazard response spectrum is based on a 2,475 year return period MCE; however, when the two percent probability of exceedance in 50 year curve is reduced by 2/3 to design level's MDE, it is similar to ten percent probability of exceedance in 50 year curve or 475 year return period. ASCE 7-05 (11.5) assigns Importance Factors (I) to structures, based on occupancy categories, which increase the lateral seismic force up to 50 percent ($I = 1.5$). Because the loss of a check structure would result in an uncontrolled release of water from the pool and thus substantial economic impacts, the importance factor for check structures is recommended to be equal to 1.25 (occupancy category III). Increasing the seismic force by 25 percent equates to roughly a 1000-year return period, which is greater than the adjacent canal (200- and 500-year return period).

4.0 POWER AND PUMPING PLANTS AND BUILDINGS

4.1 BACKGROUND INFORMATION

DWR owns 31 power and pumping plants and 308 buildings associated with the SWP. The majority of these structures were constructed in the 1960s and 1970s as part of the original SWP. Per page five of Bulletin 200 Volume IV, Power and Pumping Facilities, the applicable editions of the following codes and standards listed below were used for design of power and pumping facilities built as part of the original SWP. It appears reasonable to assume that buildings were also designed to these same codes and standards.

1. American Concrete Institute (ACI)
 - a. “Building Code Requirements for Reinforced Concrete”
 - b. “Manual of Standard Practice for Detailing Reinforced Concrete Structures”
2. American Institute of Steel Construction (AISC)
 - a. “Manual of Steel Construction”
 - b. “Specification for the Design, Fabrication and Erection of Structural Steel for Buildings”
3. American Society for Testing and Materials (ASTM)
 - a. Pertinent ASTM standards
4. American Welding Society (AWS)
 - a. “Code for Welding in Building Construction” D1.0-66
5. Pacific Coast Building Association
 - a. “Uniform Building Code”

Page six of Bulletin 200 Volume II, Conveyance Facilities states that “earthquake hazard reports were prepared for each major structure.” A major structure is a dam, or pumping or power plant. Per page five of Bulletin 200 Volume IV, Power and Pumping Facilities, the seismic design criteria for the construction of the original SWP were based on the report submitted by the Consulting Board for Earthquake Analysis (CBEA) on November 19, 1962. The CBEA’s recommendations for the design of major power and pumping plants were essentially that: (1) rigid structures (such as the substructure for a power plant) should be designed for a maximum horizontal peak ground acceleration of 0.5 g and a maximum vertical acceleration of 0.33 g acting simultaneously, and (2) flexible building structures (such as the superstructure of a power plant) should be designed using accelerations obtained from the spectral response curves included in the CBEA report and reproduced on page six of Bulletin 200 Volume IV. Based on this information, it is reasonable to assume that buildings were designed as flexible structures using accelerations obtained from the spectral response curves included in the CBEA report. This assumption will have to be verified on a building by building basis.

After the construction of the SWP in the 1960s and 1970s, various consultants reviewed the seismic vulnerability of the SWP facilities. Based on the Seismic Risk Analysis for California State Water Project, 1978, by S. A. Kiremidjian and C. H. Shah, the superstructures and substructures of power plants and pumping plants were designed for peak ground acceleration (PGA) of 0.50 g and of the switchyard equipment were designed for a PGA of 0.2 g. The study's results demonstrated that the risk of damage or failure to pumping and power plant substructures and superstructures was relatively small. However, the risk to switchyards was found to be considerably high and the consultants recommended that modifications be made to the switchyard equipment in order to reduce the risk.

After the 1975 Oroville Earthquake, DWR conducted a seismic re-analysis of the Oroville facilities including Edward Hyatt Powerplant and Thermalito powerplant using 0.25 g peak ground acceleration (DWR Bulletin 203-78, February 1979). A special Oroville Earthquake consulting board recommended re-analysis of seismic loading. However, the report does not state why the 0.25 g PGA was selected for analyzing the power plants. Based on the findings, it was concluded that the substructures of these power plants would be capable resisting the force induced by 0.25g PGA and no modification was required. However, some modifications were made to improve the seismic resistance of the power house superstructure components and intake structures.



However, some modifications were made to improve the seismic resistance of the power house superstructure components and intake structures.

Structures added to the SWP in the recent years were designed using the seismic provisions of the then current CBC. DWR engineers currently use the 2010 CBC to determine the appropriate seismic loads needed in the design of new structures.

In December of 2005, the USBR Building Safety Program performed a seismic evaluation of the William R. Gianelli Pumping-Generating Plant. Stated on page 1-8 of Part 1 of the evaluation:

“Currently, Reclamation adopts the seismic design provisions of the International Building Code (IBC, 2003) for the design of new pumping and power plants.”

The USBR still uses this approach as confirmed in June 2011 telephone conversations with structural engineers Rodney Barthel (303-445-3221) and David Kresin (303 445 3131) wherein they verified that USBR is currently using the seismic design provisions of the 2009 IBC for the design of new pumping and power plants.

The CBC is based on the IBC with amendments to specific sections made by a variety of regulatory agencies of the State of California. DWR's practice of using the CBC to determine the appropriate seismic loads is thus consistent with USBR practice.

Section 1613.1 of the 2010 CBC governs the seismic design of structures and states:

“Every structure and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7.”

The term “ASCE 7” is specifically referencing ASCE 7-05 as noted in Chapter 35 of the 2010 CBC, Referenced Standards. Thus, all earthquake (seismic) analysis is based on the provisions of ASCE 7-05.

4.2 ANALYSIS PROCEDURE

4.2.1 Design of New Structures

The seismic analysis required by Section 1613 of the 2010 CBC consists of one of three procedures permitted in ASCE 7-05. The three permitted analytical procedures are (1) Equivalent Lateral Force (ELF), (2) Modal response spectrum analysis, and (3) Seismic response history analysis. Either the modal response spectrum analysis or seismic history analysis is required for:

- Structures with horizontal and/or vertical irregularities, and
- Structures situated on sites containing peat, highly plastic clays or collapsible soil (designated Type F in ASCE Table 20.3-1).

SWP structures do not typically include horizontal and/or vertical irregularities and are typically not founded on Type F soil and thus may be designed using ELF. DWR engineers typically apply the ELF procedure to the majority of SWP structures. The ELF procedure utilizes seismic hazard maps in ASCE 7-05 that are based on a set of probabilistic maps developed by the USGS. The remainder of this section will focus on the ASCE 7-05 ELF procedure.

4.2.1.1 Background on Development of ASCE 7-05 Seismic Hazard Maps

In June of 1996, the USGS prepared new seismic hazard maps for the conterminous United States. These 1996 seismic hazard maps were referenced in the 1997 NEHRP (National Earthquake Hazards Reduction Program) Recommended Seismic Provisions for New Buildings and Other Structures (FEMA 302), and included in ASCE 7-98 and ASCE 7-02. The 1996 seismic hazard maps were updated in 2002, referenced in the 2003 NEHRP Recommended Seismic Provisions for New Buildings and Other Structures (FEMA 450), and included in ASCE 7-05. In 2008, the 2002 seismic hazard maps were updated.

The 2008 seismic hazard maps incorporated new NGA relationship equations for crustal faults in the Western United States. The 2008 seismic hazard maps were referenced in the 2009 NEHRP Recommended Seismic Provisions for New Buildings and Other Structures (FEMA 750), and served as the basis for the seismic maps in ASCE 7-10. Since June of 1996 the seismic hazard maps have been continuously available online. Chapter 35 Referenced Standards of Volume 2 of the 2010 CBC specifies the use of ASCE 7-05 and thus use of the 2002 seismic hazard maps.

The approach adopted in ASCE 7-05 is intended to provide for a uniform margin against collapse at the design ground motion. In order to accomplish this, ground motion hazards are defined in terms of MCE ground motions. The MCE ground motions are based on a set of rules that depend on the seismicity of an individual region. The design earthquake ground motions are based on a lower bound estimate of the margin against collapse inherent in structures designed to the provisions of ASCE 7-05. This lower bound was judged, based on experience, to correspond to a factor of about 1.5 in ground motion. Consequently, the design earthquake ground motion was selected at a ground shaking level that is 1/1.5 (2/3) of the MCE ground motion.

For most regions of the nation, the MCE ground motion is defined with a uniform probability of exceedance of two percent in 50 years (return period of about 2,500 years). While stronger shaking than this could occur, it was judged that in high seismic areas it would be economically impractical to design for such very rare ground motions and that the selection of the two percent probability of exceedance in 50 years as the MCE ground motion would result in acceptable levels of seismic safety for the nation.

In regions of high seismicity, such as coastal California, the seismic hazard is typically controlled by large-magnitude events occurring on a limited number of well defined fault systems. Ground shaking calculated at a two percent probability of exceedance in 50 years would be much larger than what would be expected based on the characteristic magnitudes of earthquakes on these known active faults. This is because these major active faults can produce characteristic earthquakes every few hundred years. For these regions, it is considered more appropriate to directly determine MCE ground motions based on the characteristic earthquakes of these defined faults. Values thus derived are denoted as deterministic values. In order to provide for an appropriate level of conservatism in the design process, when this approach is used, the median estimate of the deterministic ground

motion resulting from the characteristic event is multiplied by 1.5. This value is then compared to the two percent in 50 year value and the lower value is used as the MCE ground motion. This procedure effectively puts a “deterministic cap” on the MCE value equal to 150 percent of the median deterministic value. Figure 4.1 shows the location of areas where the MCE is governed by the deterministic cap.

Figure 4.1

Location of Deterministic Areas



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For the Western United States, multiplying the 2500-year uniform response spectrum curve by 2/3 generates a curve that is approximately equal to the 500-year uniform response spectrum curve; i.e. a response spectrum curve with a ten percent probability of exceedance in 50 years. Multiplying the 2500-year uniform response spectrum curve by 3/4 generates a curve that is approximately equal to the 1000-year uniform response spectrum curve; i.e. a response spectrum curve with a ten percent probability of exceedance in 100 years.

4.2.1.2 Steps Involved in the ELF Procedure

Note: All equations, sections and figure references below are from the ASCE 7-05.

The ELF procedure calculates the horizontal seismic base shear, V , in a given direction in accordance with the equation (12.8-1) shown below:

$$(Equation 12.8-1) \quad V = C_s W$$

Where,

C_s = the seismic response coefficient determined per Section 12.8.1.1 as discussed in the following sections.

W = the effective seismic weight per Section 12.7.2.

4.2.1.3 Mapped Acceleration Parameters

The parameters S_S and S_1 shall be determined from the 0.2 and 1 s spectral response accelerations (at five percent of critical damping) shown on Figs. 22-1, 22-3, 22-5, and 22-6 for S_S and Figs. 22-2, 22-4, 22-5, and 22-6 for S_1 . Values of S_S and S_1 may also be obtained directly from the USGS website based on input of site longitude and latitude.

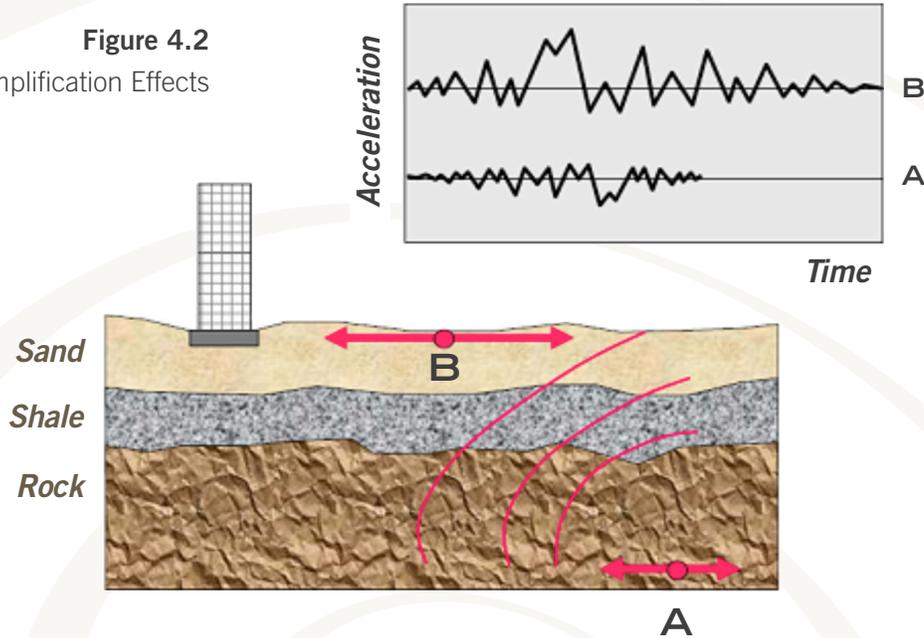
4.2.1.4 Site Classification

Based on the site soil properties, the site shall be classified as Site Class A, B, C, D, E, or F in accordance with Chapter 20. Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the authority having jurisdiction or geotechnical data determines Site Class E or F soils are present at the site. Site Class A is hard rock while Site Class F is soil that is vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils.

4.2.1.5 Site Coefficients

The values of S_S and S_1 obtained from Figs. 22-1, 22-3, 22-5, and 22-6 for S_S and Figs. 22-2, 22-4, 22-5, and 22-6 for S_1 are for sites on very firm soil. For sites on softer soil, the ground motions will be amplified. The following Figure 4.2 shows ground motion occurring in rock (lower time history) and in a softer material such as clay. At point B, the ground motions are significantly amplified over those at point A. In addition, the duration of the motion may be increased and the frequency content may change. The principal effect is that high frequency components are filtered out and longer period motions are enhanced.

Figure 4.2
Site Amplification Effects



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The site coefficients F_a and F_v selected from Tables 11.4-1 and 11.4-2 quantify the site amplification generated by the different soil types and are used in conjunction with S_S and S_1 to determine the Adjusted Maximum Considered Spectral Response Parameters S_{MS} and S_{M1} for a specific site using equations 11.4-1 and 11.4-2 shown below.

$$(Equation 11.4-1) \quad S_{MS} = F_a S_S$$

$$(Equation 11.4-2) \quad S_{M1} = F_v S_1$$

4.2.1.6 Design Spectral Response Acceleration Parameters S_{DS} and S_{D1}

The Adjusted Maximum Considered Spectral Response Parameters are multiplied by 2/3 to determine the Design Spectral Response Acceleration Parameters S_{DS} and S_{D1} as shown below.

$$(Equation 11.4-3) \quad S_{DS} = 2/3 S_{MS}$$

$$(Equation 11.4-4) \quad S_{D1} = 2/3 S_{M1}$$

4.2.1.7 Importance Factor (I)

An Importance Factor, I , shall be assigned to each structure in accordance with Table 11.5-1 based on the Occupancy Category from Table 1-1. In general, facilities that house large groups of occupants, or occupants that have reduced mobility, are assigned to Occupancy Category III and have a seismic importance factor of 1.25.



In general, facilities needed for emergency response and facilities that house significant quantities of hazardous materials are assigned to Occupancy Category IV and have a seismic importance factor of 1.5. Other occupancy types generally fall under Occupancy Categories I and II and have a seismic importance factor of 1.0.

Water and electricity from the SWP may be required for emergency response immediately following any man-made or natural disaster. Those portions of the SWP structures involved in the pumping, conveying, regulating or control of SWP water or in the generation of electricity may be needed for emergency response and should be assigned to Occupancy Category IV with a seismic importance factor of 1.5. The SWP structures that would fall into the Occupancy Category IV include power and pumping facilities, operations and control buildings, and storage facilities containing significant quantities of hazardous materials. All other SWP structures should be assigned to Occupancy Category III or II with a seismic importance factor of 1.25 or 1.0.

4.2.1.8 Response Modification Coefficient (*R*)

Past experience and observation of structure behavior following earthquakes has shown that a structure can be economically designed for a fraction of the estimated elastic seismic design forces and still maintain the basic life safety performance objective. The reason for the adequate performance of structures designed using this approach is thought to be the result of a combination of extra or reserve strength in the structural system and stable inelastic behavior of the structural elements. The inelastic behavior of a structure absorbs a significant amount of seismic energy from the system as the structure deforms during a seismic event.

The capacity of a structure to absorb energy in the inelastic range is called ductility. The Response Modification Factor, *R*, quantifies the ductility of a structure. A Response Modification Factor, *R*, shall be assigned to each structure in accordance with Table 12.2-1 based on the structure's Seismic Force Resisting System. *R* values are typically higher for those systems that have more ductility and range from a low of 1½ for Ordinary Plain Masonry Shear Walls to a high of 8 for a Steel Special Moment Frame.

4.2.1.9 Fundamental Period of the Structure (*T*)

T is permitted by Section 12.8.2 to be taken equal to T_a as calculated below:

$$\text{(Equation 12.8-7)} \quad T_a = C_t h_n^x$$

Where,

h_n = the height in feet above the base to the highest level of the structure.

Values of C_t and *x* are taken from Table 12.8-2 based on structure type.

4.2.1.10 Long-Period Transition Period (T_L)

T_L marks the transition between the constant displacement and constant velocity segments of the Design Response Spectrum. T_L is shown in Fig. 22-15 or may also be obtained directly from the USGS website based on input of site longitude and latitude.

Using these variables the seismic response coefficient C_s is then calculated per equation (12.8-2) as shown below.

$$(Equation\ 12.8-2) \quad C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)}$$

The value of C_s computed in accordance with Eq.12.8-2 need not exceed the following:

$$(Equation\ 12.8-3) \quad C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \quad \text{for } T \leq T_L$$

$$(Equation\ 12.8-4) \quad C_s = \frac{S_{D1}T_L}{T^2\left(\frac{R}{I}\right)} \quad \text{for } T > T_L$$

C_s shall not be less than

$$(Equation\ 12.8-5) \quad C_s = 0.044S_{D1} \geq 0.01$$

In addition, for structures located where S_1 is equal to or greater than 0.6g, C_s shall not be less than

$$(Equation\ 12.8-6) \quad C_s = \frac{0.5S_1}{\left(\frac{R}{I}\right)}$$

The horizontal seismic base shear, V , is then calculated per equation (12.8-1) as shown below.

$$(Equation\ 12.8-1) \quad V = C_s W$$

The vertical seismic force, E_v , is then calculated per equation (12.14-6) as shown below.

$$(Equation\ 12.14-6) \quad E_v = 0.2S_{DS}D$$

The above steps for the ELF procedure shall be reviewed and revised as new versions of the CBC become available.

4.2.2 Evaluation of Existing Structures

Included in the CBC are provisions that encourage or require designs with features important for good seismic performance, including regular configuration, structural continuity, ductile detailing, and materials of appropriate quality. Many existing structures were constructed without these features and contain characteristics such as unfavorable configuration and poor detailing that preclude application of the CBC provisions for their seismic evaluation. ASCE/SEI 31-03 "Seismic Evaluation of Existing Buildings" (ASCE 31-03) was developed specifically to assist in the evaluation of the seismic performance of existing buildings, and can be used for other types of structures.

The USBR Building Safety Program followed the ASCE 31-03 procedures in their seismic evaluation of the William R. Gianelli Pumping-Generating Plant (see page 1-2 of the evaluation). On page 1-8 of the USBR Building Safety Program seismic evaluation of the William R. Gianelli Pumping-Generating Plant, it is stated that

"Given the longer service-life and essential function of power plants as a lifeline, the Program uses an evaluation seismic event having a ten percent probability of being exceeded in 100 years (ten percent in 100 year) or a return period of approximately 1000 years for the evaluation of the plant's primary structures. This seismic loading is approximately equivalent to $\frac{3}{4}$ MCE. The plant's ancillary structures, however, are evaluated to the $\frac{2}{3}$ MCE or the ten percent in 50 year event."

The primary structures are essential for the facility's continued operation following a design level seismic event and the ancillary structures are not required to be operational for the facility's continued function during the period immediately following a design level seismic event.

Per the CBC, new structures' seismic design utilizes an equivalent lateral horizontal force (horizontal base shear, V) as described in the preceding paragraphs. The base shear is representative of the force that the structure is expected to resist, but the structure displacements calculated based on this force are significantly less than the actual displacements of the structure during a design earthquake.

ASCE 31-03 uses an equivalent displacement methodology that imposes a pseudo lateral force on the structure to obtain "actual" structure displacements generated by a design earthquake. These displacements are the expected actual displacements of the structure in the yielded state. A modification factor C is used to adjust the pseudo lateral force to a value that results in attaining the "actual" displacements. The "actual" force demands on components are obtained by application of a modification factor (m factor) which is based on the ductility and required seismic performance level of the component. Each component is then analyzed for its ability to withstand these actual forces.

The first step in the ASCE 31-03 evaluation process is to determine the required performance level for a structure. Two performance levels for both structural and nonstructural components are defined in ASCE 31-03: (1) Life Safety (LS) and (2) Immediate Occupancy (IO). The LS performance level allows a level of damage to both structural and nonstructural components during a design earthquake, such that: (a) partial or total structural collapse does not occur, and (b) damage to nonstructural components is non-life-threatening. The IO performance level allows a level of damage to both structural and nonstructural components during a design earthquake, such that: (a) the damage is not life threatening, so as to permit IO of the structure after a design earthquake, and (b) the damage is repairable while

the structure is occupied. For both performance levels, the seismic demand is based on a fraction of the MCE spectral response acceleration values obtained from the 2002 seismic hazard maps in ASCE 7-05. SWP structures that have been classified Occupancy Category IV should be evaluated to the IO performance level. All other SWP structures should be evaluated to the LS performance level.

The second step in the ASCE 31-03 evaluation process is to determine the level of seismicity for a structure based upon S_{DS} and S_{D1} . The level of seismicity of a structure shall be defined as low, moderate, or high in accordance with ASCE’s Table 2-1 (Table 4.1 in this report).

Table 4.1 Levels of Seismicity Definitions Evaluation Requirements (ASCE 31-03, Table 2-1)

Level of Seismicity ¹	S_{DS}	S_{D1}
Low	< 0.167g	< 0.067g
Moderate	≥ 0.167g < 0.500g	≥ 0.067g < 0.200g
High	≥ 0.500g	≥ 0.200g

¹Sites with S_{DS} and S_{D1} values in different levels of seismicity shall be classified as moderate.

Where,

S_{DS} = Design short-period spectral response acceleration parameter (Sec. 3.5.2.3.1)

S_{D1} = Design spectral response acceleration parameter at a one-second period (Sec. 3.5.2.3.1)

The third step in the ASCE 31-03 evaluation process is to classify the structure as one of 24 common building types (CBT). The CBT’s are standard designations that capture most of the standard structural configurations. Checklists for on-site inspections are provided for each CBT. The appropriate checklist is selected based upon the required performance level and the level of seismicity. The CBT checklists address the structural issues for the structure, while geologic site and foundation issues are addressed in separate checklists. The checklists focus on the features required for the desired level of structure performance and are substantially completed during the on-site inspection process. The evaluator answers each of the checklist items with a compliant (C), noncompliant (NC), or not applicable response (NA). An NC response to any checklist item indicates a potential seismic deficiency, and the evaluator must then analyze that particular feature further to see if a deficiency actually exists.

The ASCE 31-03 seismic evaluation process uses a three-tiered approach to identify potential seismic deficiencies in a structure. The purpose of a Tier 1 Evaluation is to identify quickly structures that comply with the provisions of ASCE 31-03 and potential deficiencies using checklists. The Tier 2 evaluation requires a linear elastic analysis to evaluate any potential deficiencies identified in the Tier 1 evaluation. Tier 1 and Tier 2 evaluations have the potential for being conservative because of the simplifying assumptions involved in their application. A Tier 3 evaluation is a more detailed analysis that is conducted if deficiencies remain after the Tier 2 evaluation or if a less conservative analysis is desired. ASCE 31-03 recommends that Tier 3 evaluations be performed using FEMA 356 Prestandard and Commentary for the Seismic Rehabilitation of Buildings. ASCE 31-03 requires that the FEMA 356 force levels be multiplied by 0.75 when used for a Tier 3 evaluation. FEMA 356 became ASCE 41-06 “Seismic Rehabilitation of Existing Buildings”.

The more detailed and presumably less conservative evaluation completed in a Tier 3 analysis may reveal that structures or structure components identified by Tier 1 and/or Tier 2 evaluations as having seismic deficiencies are satisfactory to resist seismic forces. The USBR Building Safety Program seismic evaluation of the William R. Gianelli Pumping-Generating Plant performed a Tier 3 evaluation on all primary structures using the provisions of FEMA 356. It is recommended that DWR adopt this type of approach and perform Tier 3 evaluations on all Occupancy Category IV structures when evaluation is required.

A rehabilitation strategy will have to be developed to address any seismic deficiencies remaining after a Tier 3 analysis. One or more of the rehabilitation strategies listed below may be utilized:

- Local modification of components.
- Removal or reduction of existing irregularities.
- Global structural stiffening.
- Global structural strengthening.
- Mass reduction.
- Seismic isolation.
- Supplemental energy dissipation.

A structure, or any of its components, is considered to be structurally adequate if it complies with the requirements of ASCE 31-03.

4.3 RECOMMENDED SEISMIC CRITERIA FOR DWR FACILITIES

Seismic loading criteria for buildings and pumping and power plants vary depending on their functionality in an emergency and during the regular operational period. Many of the buildings and pumping and power plants are expected to be operated almost continuously to avoid long interruption of water supply. They are expected to be functional during or immediately after a large earthquake. Therefore, the seismic loading criteria and the performance criteria should be higher for these facilities.

When selecting seismic loading criteria for a building many factors should be considered, including operation aspect of the facility in relation to emergency activities, potential interruption of water delivery and its impact to the economy and environment, and repair cost and time.

4.3.1 Design of New Structure

All critical facilities, such as pumping and power plants that are expected to be functional during an emergency and control buildings, should be assigned to the Occupancy Category IV with a seismic importance factor (I) of 1.5. The other facilities can be assigned to the appropriate Importance Factor based on the Occupancy Category from Table 1-1 in ASCE 7-05.

Seismic design load for the critical facilities is recommended to be larger than that for the regular buildings to provide additional safety factor. Generally, the ASCE design response curve is approximated to about the 500-year response spectrum curve, which is recommended for DWR non-critical buildings. However, for DWR critical buildings, it is recommended to use the design curve of 1000-year return period and the important factor of 1.5.

The above recommendations can be implemented using the ASCE 7-05 seismic maps by adding the factor α to equations 11.4-3 and 11.4-4 as shown below.

$$(Equation\ 11.4-3') \quad S_{DS} = \alpha S_{MS}$$

$$(Equation\ 11.4-4') \quad S_{D1} = \alpha S_{M1}$$

Where,

$\alpha = 3/4$ for Occupancy Category IV structures and $\alpha = 2/3$ for all other structures

Similarly, if site specific design curves are necessary, the ASCE equation in Chapter 21 can be modified as follows:

$$(Equation\ 21.3-1') \quad S_a = \alpha S_{aM}$$

Where,

$\alpha = 3/4$ for Occupancy Category IV structures, and

$\alpha = 2/3$ for all other structures

4.3.2 Future Evaluation of Existing Structures

ASCE 31-03 references the 1996 seismic hazard maps for the conterminous United States developed by the USGS that are in ASCE 7-02. The 1996 seismic hazard maps defined the MCE ground motion as a seismic event with a probability of exceedance of two percent in 50 years (return period of about 2500 years). In 2002, the maps were updated and included in ASCE 7-05. The 2002 seismic hazard maps were updated in 2008 to include the 2008 NGA relationships for crustal faults in the Western United States. The 2008 seismic hazard maps were modified and included in ASCE 7-10. Per ASCE 7-10, Figure 22-1, the modified maps in ASCE 7-10 incorporate the following:

- A target risk of structural collapse equal to one percent in 50 years based on a generic structural fragility.
- A factor of 1.1 to adjust from a geometric mean to the maximum response regardless of direction.
- Deterministic upper limits imposed near large, active faults, which are taken as 1.8 times the estimated median response to the characteristic earthquake for the fault (1.8 is used to represent the 84th percentile response), but not less than 150 percent.

The maps in ASCE 7-10 are thus based on a different set of assumptions than the 1996 or 2002 seismic hazard maps and are not appropriate for use with ASCE 31-03. For this reason, it is recommended that the 2002 seismic hazard maps included in ASCE 7-05 be used with ASCE 31-03, as they are consistent with the assumptions made in the document until ASCE 31-03 is updated to specifically reference the 2008 seismic hazard maps and/or ASCE 7-10.

Based on the current standard of practice by DWR and other agencies, it is recommended that DWR adhere to the provisions of ASCE 31-03 to evaluate all existing structures when evaluation is required. For critical facilities (Occupancy Category IV) structures, use ASCE 7-05 design curve with the factor $\alpha = 3/4$ in equations 11.3-3, 11.3-4, and 21.3-1; and for other facilities Use ASCE 7-05 design curve with the factor $\alpha = 2/3$.

4.4 MECHANICAL AND ELECTRICAL EQUIPMENT

4.4.1 Background

In the past, DWR required that mechanical and electrical equipment and its anchorage be designed to withstand the stresses created by seismic loads. For mechanical equipment, DWR specified that a horizontal seismic force of magnitude $1.0W_p$ be applied concurrently with a vertical seismic force of magnitude $1.0W_p$ in the direction which produces the most severe stresses on the equipment with a weight of W_p . In recent years, this requirement was met in most cases. However, this requirement also proved difficult to meet in some situations that required anchoring of equipment to an existing structure and for free-standing structures such as a gantry crane that rolls on tracks.

Current DWR electrical specifications allow the contractor to use the Static Coefficient Analysis method of The Institute of Electrical and Electronics Engineers, Inc. (IEEE, 2004) Standard 344 "Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations". Using this method, the horizontal seismic force on each component of the equipment is obtained by multiplying W_p by the maximum acceleration of the appropriate response spectrum curve times a static coefficient of 1.5. The static coefficient of 1.5 has been established from experience to take into account the effects of multi-frequency excitation and multimode response for linear frame-type structures. This relationship can be stated mathematically as shown below:

$$F_p = 1.5a_{\max}W_p$$

The IEEE does not include any procedure or discussion of how to obtain appropriate design spectra, but provides the above equation to calculate the seismic force after developing the design spectra.

4.4.2 ASCE 7-05 Requirements

Section 13.3.1 of ASCE 7-05 requires mechanical and electrical equipment to be designed to resist the seismic forces in the horizontal and vertical directions acting on the equipment's center of gravity. Equations 13.3-1, 13.3-2 and 13.3-3 are used to calculate the horizontal seismic design force, F_p .

(Equations 13.3-1, 13.3-2, and 13.3-3)

$$F_p = 0.3S_{DS}I_pW_p < \frac{0.4a_pS_{DS}}{\left(\frac{R_p}{I_p}\right)} \left(1 + 2\frac{z}{h}\right)W_p < 1.6S_{DS}I_pW_p$$

Where,

a_p = component amplification factor that varies from 1.00 to 2.50 from Table 13.6-1.

I_p = component importance factor per Section 13.1.3.

W_p = component operation weight.

R_p = component response modification factor from Table 13.6-1 that varies from 1 to 12.

z = height in structure of point of attachment of component with respect to the base. For items at or below the base, z shall be taken as 0. The value of z/h need not exceed 1.0.

h = average roof height of structure with respect to the base.

Section 13.3.1 of ASCE 7-05 requires that a vertical force of magnitude $\pm 0.2SDSW_p$ be applied concurrently with the horizontal force F_p in the direction which produces the most severe stresses on the equipment.

4.4.3 Recommended Procedure

The following recommendations are provided to calculate seismic forces on electrical and mechanical equipment:

- Adhere to the requirements of ASCE 7-05 Section 13.2.1 and require that mechanical and electrical equipment manufacturers provide certification that components are seismically qualified.
- Use Section 13.3.1 of ASCE 7-05 to determine the magnitudes of horizontal and vertical seismic forces.
- Use $I_p = 1.5$ for mechanical equipment and 1.75 for electrical equipment in Occupancy Category IV for critical facilities as discussed in Section 4.3.5 of this report. A higher importance factor is used for electrical equipment as there is potential for fire and explosion hazards.
- Retain the discretion to modify the required design seismic force level stipulated above in situations where new equipment is to be anchored to an existing structure. When the seismic loading is reduced below the above recommendations, written approval of a senior staff is required.

5.0 BRIDGES

5.1 EXISTING DWR FACILITIES

There are 329 bridges crossing the SWP. DWR owns only 166 of these bridges while local, state, railroad, and federal agencies own the remaining bridges. Approximately 96 DWR bridges cross the California Aqueduct main stem and the remainder cross dams, spillways, creeks, roadways, forebays, afterbays, and reservoirs as well as the Coastal, South Bay, East Branch and West Branch canals.

Approximately 72 percent of DWR owned bridges were designed and constructed before 1970, as shown in Figure 5.1. As-built plans explicitly indicate that DWR bridges were designed for truck live loading as specified in the then current AASHTO Standard Specifications for Highway Bridges. AASHTO editions prior to 1941 did not address seismic design. The 1941 and 1949 editions mentioned seismic loading, but simply stated that structures shall be proportioned for earthquake stresses with no guidance or criteria as to how the earthquake forces were to be determined or applied. In summary, it is not clear to what seismic criteria and/or loading level DWR bridges were designed.

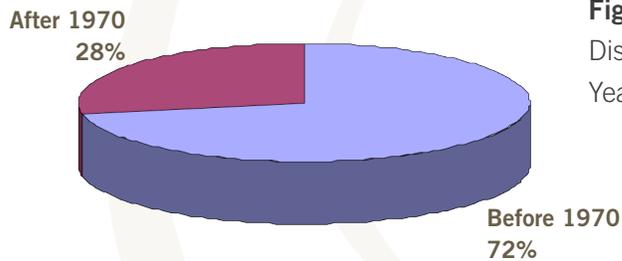


Figure 5.1

Distribution of DWR Owned Bridges by Construction Year (Total of 166 DWR Owned Bridges)

5.2 DESIGN CRITERIA USED BY DWR AND OTHER AGENCIES

Almost all 329 bridges were constructed in the 1960's. As indicated in the as-builts, these bridges were designed according to the American Association of State Highway and Transportation Officials' (AASHTO) Standard Specifications for Highway Bridges (1949). Since seismic standards for bridge design were not developed until after 1975, these bridges were not designed to withstand current seismic design loads. However, this does not indicate that they have no resistance at all to seismic loads. Of the 166 bridges owned by DWR, the geometry, bridge types and configurations allow many of the bridges a measure of seismic resistance.

5.2.1 AASHTO Bridge Standard Specifications (2002) – Division I

Provisions of the AASHTO Standard Specifications apply to bridges of conventional steel, concrete girder, and box girder construction with spans not exceeding 500 feet. These provisions do not cover suspension bridges, cable-stayed bridges, arch type and movable bridges. Seismic design is usually not required for buried type (culvert) bridges.

Provisions under Standard Specifications require that small to moderate earthquakes be resisted by the bridges without significant damage and that all or part of the bridge not collapse under large earthquakes. If damage does occur, it should be readily detectable and accessible for inspection and repair.

The following is an outline of the steps to be followed to determine seismic hazard and to complete seismic design:

- **Acceleration Coefficient (A):** The coefficient is obtained from a map of horizontal acceleration, (expressed as percent of gravity) in rock with ten percent exceedance in 50 years (15 percent exceedance in 75 years). This corresponds to a return period of approximately 475 years. Special studies to determine site- and structure-specific acceleration coefficients shall be performed if: 1) The site is located close to an active fault; 2) Long duration earthquakes are expected in the region; or 3) The importance of the bridge is such that a longer exposure period (and therefore return period) should be considered.
- **Importance Classification (IC):** An Importance Classification is assigned for all bridges with an (A) greater than 0.29. This classification is used to determine the Seismic Performance Category (SPC) in the following step. IC of I is assigned for essential bridges and IC of II for all others. Bridges shall be classified based on social/survival and security/defense requirements. The determination of the IC of a bridge is necessarily subjective. Bridges on routes to critical facilities such as hospitals, police, fire stations, and communication centers must continue to function and should be classified as essential. In addition, a bridge that has the potential to impede traffic if it collapsed onto an essential route should also be classified as essential. An additional consideration would be the Average Annual Daily Traffic (ADT). The importance classification depends on the range of options available and the possibility of a bridge being in parallel or series with other bridges in a roadway network.
- **Seismic Performance Categories (SPC):** Four performance categories are defined from A through D. Each bridge is assigned to one of four performance categories based on the Acceleration Coefficient (A) and the Importance Classification (IC), as shown in Table 5.1.

Table 5.1 Seismic Performance Category (SPC)

Acceleration Coefficient	Importance Classification	
	I	II
A	I	II
$A \leq 0.09$	A	A
$0.09 < A \leq 0.19$	B	B
$0.19 < A \leq 0.29$	C	C
$0.29 < A$	D	C



- The SPC controls the degree of complexity and sophistication of the analysis and design requirements. Sections five through seven of Division I-A detail the requirements for bridges design in seismic performance categories A, B, C, and D. Regardless, the SPC and the bridge configuration will also control the selection and method of seismic analysis. Standard bridges with two to six

spans in any SPC can be analyzed based on uniform load method or single-mode spectral. Non standard bridges with two or more spans will require a more rigorous analysis such as multimode spectral or time history methods.

5.2.2 AASHTO LRFD Bridge Design Specifications – 2007

Article 3.10.3 of AASHTO Load Resistance Factor Design (LRFD) Bridge Design Specifications establishes three importance levels with respect to seismic design: critical, essential, and other bridges. The provisions are for conventional slab, beam girder, box girder, and truss superstructure bridges with spans not exceeding 500 ft. Seismic effects for box culverts and buried structures need not be considered unless they cross active faults.

The seismic hazard and design is being determined in a, more or less, similar methodology as the Standard Specifications described below.

- *Acceleration Coefficient (A)*: Acceleration Coefficient is determined from the contour maps prepared by USGS for NEHRP following provisions for Development of Seismic Regulations for New Buildings. The coefficient is based on a uniform risk model of seismic hazard with ten percent exceedance in 50-year period. If the bridge is close to an active fault or a long-duration earthquake is expected and the bridge is considered important, special studies should be performed to determine site and structure specific acceleration coefficients.
- *Importance Categories*: Importance category is established as critical, essential and other bridges. Please note that the definitions of critical and essential by AASHTO are different from the definitions provided by FEMA for pipelines. The classifications are mainly based on social/survival and security/defense requirements with consideration to possible future changes in conditions and requirements. Essential bridges should be open to emergency traffic immediately after the design earthquake (475-year return period). Critical bridges must be open to all traffic after design earthquake and usable by emergency vehicles after a large earthquake (2500-year return period).
- *Seismic Zones*: Each bridge is assigned to one of four seismic zones based on acceleration coefficient as shown in Table 5.2. Section 3.10.9 details the requirements for calculation of design seismic forces for bridges in each of the seismic zones.

Table 5.2 Seismic Zones*(From Table 3.10.4-1 in AASHTO LRFD Bridge Design Specifications 2007)*

Acceleration Coefficient	Seismic Zone
A	
$A \leq 0.09$	1
$0.09 < A \leq 0.19$	2
$0.19 < A \leq 0.29$	3
$0.29 < A$	4

- **Site Coefficient (S):** The Seismic Load is modified by the Site Coefficient (S) based on four soil profile types as defined in Table 5.3 below. Type I soil is characterized by a shear wave velocity greater than 2500 ft/sec while Type IV soil has a shear wave velocity less than 500 ft/sec.

Table 5.3 Soil Profile Types

Site Coefficient	Soil Profile Type			
	I	II	III	IV
S	1.0	1.2	1.5	2.0

After 2007, AASHTO published a standalone guide specifications for LRFD seismic bridge design. The first edition was published on 2009 as discussed in the following section.

5.2.3 AASHTO Guide Specifications for LRFD Seismic Bridge Design – 2009

These Guide Specifications are considered the first major changes of the current seismic design requirement under AASHTO LRFD Bridge Design Specifications. The development of this guide is basically an effort to combine and supplement existing seismic design procedures in AASHTO Standard Specifications Division I-A, NCHRP 12-49 guidelines, South Carolina Department of Transportation Specifications, Caltrans Seismic Design Criteria and others into a single document that could be used at a national level to design bridges for seismic effects. Key features under this guide adopted the seven percent in 75-year design event for development of a design spectrum, the NEHRP site classification system, and site factors in determining the response spectrum ordinates.

The scope of these Guide Specifications covers seismic design for conventional bridge types and applies to noncritical and non-essential bridges. Critical and essential bridges are defined by AASHTO LRFD Bridge Design Specifications 2007. A bridge should be classified as critical or essential if 1) It is required to be open to all traffic once inspected after the design earthquake and usable by emergency vehicles, 2) It should be opened to emergency vehicles within days after earthquake event, and 3) It is formally designated as critical for a defined local emergency plan.



Suspension, cable-stayed, truss, arch type, and movable bridges are not covered by these guidelines. Seismic effects for box culverts and buried structures should not be considered. However, unstable ground conditions such as liquefaction, landslides, and fault displacements must be considered.

The seismic ground shaking hazard is characterized using an acceleration response spectrum and is determined using the general procedure or site-specific procedure.

In the general procedure, the spectral response parameters are determined using the 2006 USGS/AASHTO Seismic Hazard Maps which depict the probabilistic ground motion and spectral response for seven percent probability of exceedance in 75 years (which is approximately equivalent to a 1000-year return period). Spectral parameters from the USGS/AASHTO Seismic

Hazard maps are for a soft rock/stiff soil condition, defined as Site Class B. It needs to be adjusted for local site effects.

The site-specific procedure consists of site-specific hazard analysis, which should be performed if new information about active seismic sources becomes available, the site is Site Class F, the bridge is classified as critical or essential, or if a higher degree of confidence of meeting the seismic performance is desired.

The guidelines establish four Seismic Design Categories (SDC): A through D based on the one-second period design spectral acceleration for the design earthquake (S_{D1}). Each bridge is assigned to one of the four categories and to be designed with minimum requirements under each category. The complexity of analysis and design is minimal for SDC-A and increases with each category from SDC-B to SDC-D.

5.2.4 Caltrans Bridge Seismic Design Criteria (SDC)

Caltrans was the first organization within the United States to develop specific seismic design criteria for bridges. In 1940, Caltrans published its first general code requirement. Based on Caltrans' Memo to Designers (MTD 20-1), bridges are categorized as either Important or Ordinary based on the desired level of seismic performance.

The Ordinary Category is further divided into two classifications - Standard and Non Standard. The seismic design criteria for Ordinary Standard Bridges are contained in the SDC. The seismic design criteria for Ordinary Standard Steel Bridges are contained in the Caltrans Guide Specifications for Seismic Design of Steel Bridges (GSSDSB). The seismic design criteria for Important Bridges and Ordinary Non-Standard Bridges need be developed on a project-specific basis to address their non-standard features.

The SDC defines the bridge as Ordinary Standard if it meets all of the following requirements:

- Span lengths less than 300 feet.
- Constructed of normal weight concrete girder, and column or pier elements.
- Horizontal members either rigidly connected, pin connected, or supported on conventional bearings (isolation bearings and dampers are considered nonstandard components)
- Foundations supported on spread footing, pile cap with piles, or pile shafts.
- Soils those are not susceptible to liquefaction, lateral spreading, or scour.
- Bridge systems with a fundamental period greater than or equal to 0.7 seconds in the transverse and longitudinal directions of the bridge.

The SDC categorized the bridge as Important if (1) the bridge is required to provide post earthquake life safety; such as access to emergency facilities, (2) time required for restoration of functionality following closure would create a major economic impact or (3) if a local emergency plan formally designates the bridge as critical.

A bridge's category and classification will determine its seismic performance level and which method is used for estimating the seismic demands and structural capacities. Earthquake motions are developed by considering the relationship of the site to active faults, the seismic response of the soils at the site, and the dynamic response characteristics of the bridge as specified in the SDC.

Seismic demand is represented using an elastic five percent damped response spectrum. The Design Spectrum (DS) is defined as the greater of (1) a probabilistic spectrum obtained from the 2008 USGS Seismic Hazard Map for five percent in 50 years probability of exceedance (or 975-year return period), (2) a deterministic spectrum based on the largest median response resulting from the maximum rupture (corresponding to M_{max}) of any fault in the vicinity of the bridge site, or (3) a statewide minimum spectrum defined as the median spectrum generated by a magnitude 6.5 earthquake on a strike-slip fault located 12 kilometers from the bridge site. A detailed discussion of the development of both the probabilistic and deterministic design spectra as well as possible adjustment factors is given in Appendix B of the SDC.

5.2.5 Federal Highway Administration (FHWA) Seismic Retrofitting Manual for Highway Bridges – 2006

In 2006, the FHWA published its revised Seismic Retrofitting Manual for Highway Bridges. The manual recommends a performance-based methodology for retrofitting highway bridges. It defines different performance expectations for bridges of varying importance while subject to different levels of seismic hazard. Four seismic performance levels are defined as follows:

Performance Level 0 (PLO): No minimum level of performance is recommended.

Performance Level 1 (PL1): Life safety. Significant damage is sustained during an earthquake and service is significantly disrupted, but life safety is assured. The bridge may need to be replaced after a large earthquake.

Performance Level 2 (PL2): Operational. Damage sustained is minimal and full service for emergency vehicles should be available after inspection and clearance of debris. Bridge should be repairable with or without restrictions on traffic flow.

Performance Level 3 (PL3): Fully Operational. No damage is sustained and full service is available for all vehicles immediately after the earthquake. No repairs are required.

The manual goes on and provides more details for defining minimal, significant, and sustained damages. It is worth noting that the performance levels are varying with level of earthquake ground motion, bridge importance and anticipated service life (ASL). Two ground motion levels (lower level – 100 year return period and upper level – 975 year return period), two importance classifications (Standard and Essential), and three service life categories (ASL 1, 2 and 3) are defined.

5.3 RECOMMENDED SEISMIC DESIGN CRITERIA FOR DWR BRIDGES

The following sections provide seismic loading criteria for new and existing bridges. Special considerations are given to the assessment of existing bridge performance under the recommended seismic loading.

5.3.1 New Bridges

Design and construction of new bridges shall be based on the most current codes. At this time, Caltrans Bridge Design Specifications (AASHTO – LRFD Bridge Design Specifications 2007 with California Amendments) and Caltrans Seismic Design Criteria are the most current specifications. The seismic loading criteria for bridges shall be the maximum of (1) required seismic loading as specified in Caltrans SDC or (2) recommended seismic loading for the California Aqueduct or other SWP facilities near the location of the bridge under consideration.

Bridges will be classified as Critical, Essential, or Other. Accordingly, each bridge will be assigned an importance factor based on life safety consideration and access to emergency facilities such as hospitals, fire stations, police stations, and communication centers. As a minimum performance requirement, bridge collapse shall not be permitted under any classification. See Table 5.4 below for recommended seismic loading and corresponding performance criteria for each classification.

Table 5.4 Seismic Loading Criteria for DWR Owned Bridges - New Design

Bridge Importance Classification	Seismic Loading	Performance Criteria
Important (Functional Level)	Project Specific	No collapse. Minimal damage. Must be open to <u>all</u> traffic almost immediately after the design earthquake.
Important (Safety Level)	Approximately 1000-2000 years	No collapse. Repairable damage. Limited access possible within days of earthquake. Full service is restorable within months.
Ordinary	The greater of: (1) A probabilistic spectrum based on a five percent in 50 years probability of exceedance (or 975 year return period); (2) A deterministic spectrum based on the largest median response resulting from the maximum rupture (corresponding to M_{max}) of any fault in the vicinity of the bridge site; (3) A statewide minimum spectrum defined as the median spectrum generated by a magnitude 6.5 earthquake on a strike-slip fault located 12 kilometers from the bridge site.	No collapse. Significant and potentially unreparable damage. Extended closure to repair is tolerated.

- **Minimal Damage:** Essentially elastic performance. Includes minor inelastic response and narrow flexural cracking in concrete. Permanent deformations are not apparent and repairs can be made under non-emergency conditions with the possible exception of superstructure expansion joints which may need removal and temporary replacement.
- **Repairable Damage:** Damage that can be repaired with a minimum loss of functionality. Inelastic response may occur, resulting in concrete cracking, reinforcement yield, and minor spalling of cover concrete. The extent of damage should be sufficiently limited so that the structure can be essentially restored to its pre-earthquake condition without replacing reinforcement or structural members. Repair should not require closure. Permanent offsets are small and there is no collapse.

- *Significant Damage*: A minimum risk of collapse, but damage that could require closure to repair. Includes permanent offsets and cracking, yielded reinforcement, and major spalling of concrete, which may require closure to repair. Partial or complete replacement of columns may be required. Beams may be unseated from bearings but no span should collapse. Similarly, foundations are not damaged except in the event of large lateral flows due to liquefaction, in which case inelastic deformation in piles may be evident.

5.3.2 Existing Bridges

For existing bridges, it is recommended to adopt the methodology developed by Caltrans in their Memorandum to Designers 20-4 in conjunction with Caltrans SDC. To avoid any interruption of operations, the seismic loading criteria for an existing bridge should also satisfy the seismic loading criteria of adjacent SWP facilities.

The ground motion for Ordinary Bridges shall be based on a design spectrum as defined in the Caltrans' SDC. The ground motion at the bridge site is dependent upon the earthquake magnitude, fault type, geology, and distance between the earthquake source and the site.

The ground motions for Important Bridges must be determined probabilistically. These determinations will be made on a project-specific basis and will be incorporated into the Important Bridge design criteria.

6.0 UTILITY OVERCROSSINGS

6.1 EXISTING DWR FACILITIES

Based upon the Operations and Maintenance Condition Assessment Program (CAP) inventory of May 2011, the SWP includes 357 utility crossings. These crossings are both publicly and privately owned and include natural gas (118), oil (75), and water (164) pipelines. Most utilities cross the canal above the concrete lining or are located within the bridge superstructures. Supports for the pipelines consist of piers that are typically located mid-span while other utilities are designed to free span the width of the canal. Pipe diameters range between 8 and 18 inches for oil, 2 and 60 inches for water, and are generally 26 inches for natural gas. There are also a few private utility pipelines that run underneath the canal.

The primary issue with utility overcrossings is uncontrolled spillage into the canal system causing water contamination. This can potentially interrupt water delivery for a long period of time. Prolonged pipe repair is a secondary matter.

In past earthquakes, buried pipelines behaved reasonably well to seismic forces, except when they crossed an active fault line or were located in liquefiable soils. However, soil displacements are known to cause pipe damage. Both liquefaction and the undermining of the soils as a result of a canal rupture could cause soil displacement and subsequent pipe failure.

6.2 DESIGN CRITERIA USED BY DWR AND OTHER AGENCIES

No seismic design criteria for utility overcrossings could be located in DWR publications, design files, or contract documents.

American Society of Mechanical Engineers (ASME) B31.8, *Gas Transmission and Distribution Piping Systems* (ASME B31.8), is the design code for pressurized gas pipes. The seismic design criteria are not defined, and it lacks specific seismic provisions. ASME B31.8 does require consideration of flexibility in the pipelines and its components where they cross known fault zones or are located in high seismic areas. The general provisions state that unstable ground and earthquake induced stresses need to be addressed, but do not indicate methods or references used to determine seismic loading.

6.2.1 Federal Emergency Management Agency

FEMA 233 - *Earthquake Resistant Construction of Gas and Liquid Fuel Pipeline Systems Serving, or Regulated by, the Federal Government* does not contain explicit requirements for seismic design, but does state that the primary concern for buried pipelines is the ability to accommodate abrupt ground distortions or differential displacements (ASCE 1984). It reports that strong and ductile steel pipelines withstood ground shaking but were unable to resist the large permanent ground deformations generated by faulting and ground failures during the 1971 San Fernando earthquake.

The American Lifeline Association Guideline and Commentary for Assessing the Performance of Oil and Natural Gas Pipelines Systems is not a code but a methodology designed to evaluate natural

hazards and human threats. Seismic design criteria utilize national hazard maps to differentiate hazard levels for earthquakes and are categorized as:

- Low (PGA < 0.15 g)
- Medium (0.15 ≤ PGA ≤ 0.5 g)
- High (PGA > 0.5 g)

The design ground motions are based on a 2,475 year return period or a two percent probability of exceedance in 50 years. There is a possibility that gas utility and pipeline companies used seismic hazard levels other than what is specified in FEMA 233.

ASCE 7-05 – *Minimum Design Loads for Buildings and Other Structures* includes hazard maps based on a 2,475 year return period (five percent damping) along with soil profiles used to determine the MCE. The design earthquake is calculated by multiplying the MCE by 2/3 which results in an approximate 500 year return period. Overcrossing seismic design forces are based on Chapter 13 of *Seismic Demands on Nonstructural Components*, which includes ASME B31 distribution piping, and is calculated using the following:

- Design spectral acceleration for short period (0.2s).
- Amplification factors.
- Response factors depending on pipe joint type.
- Importance Factors.
- Operating weight.

Seismic forces on pipelines should be applied to two orthogonal horizontal directions in addition to a concurrent vertical seismic force based on 20 percent of the design spectra acceleration. As a side note, the National Fire Protection Association (NFPA), the authority on fire and electrical safety, implements ASCE 7-05 for use in fire suppression piping systems.

6.3 RECOMMENDED SEISMIC CRITERIA FOR OVERCROSSINGS

Because of the lack of specific seismic design criteria related to utility pipelines with hazardous materials, it is recommended that the ASCE 7-05 *Minimum Design Loads for Buildings and Other Structures* be used to establish seismic loading for utility overcrossings. ASCE 7-05 covers a wide range of structures and related components, utilizes the most current seismic hazard maps, undergoes regular updates, and is incorporated into other agency codes. New DWR water transmission pipelines also use ASCE to design for seismic forces. ASCE 7-05 (13.1.3) assigns importance factors (I_p) to nonstructural components, based on contents of the component, which increases the lateral seismic force up to 50 percent ($I_p = 1.5$). When the overcrossing pipelines contain hazardous material, the importance factor is 1.5. Increasing the seismic force by 50 percent would be equated to a 2,475 year return period or MCE event.

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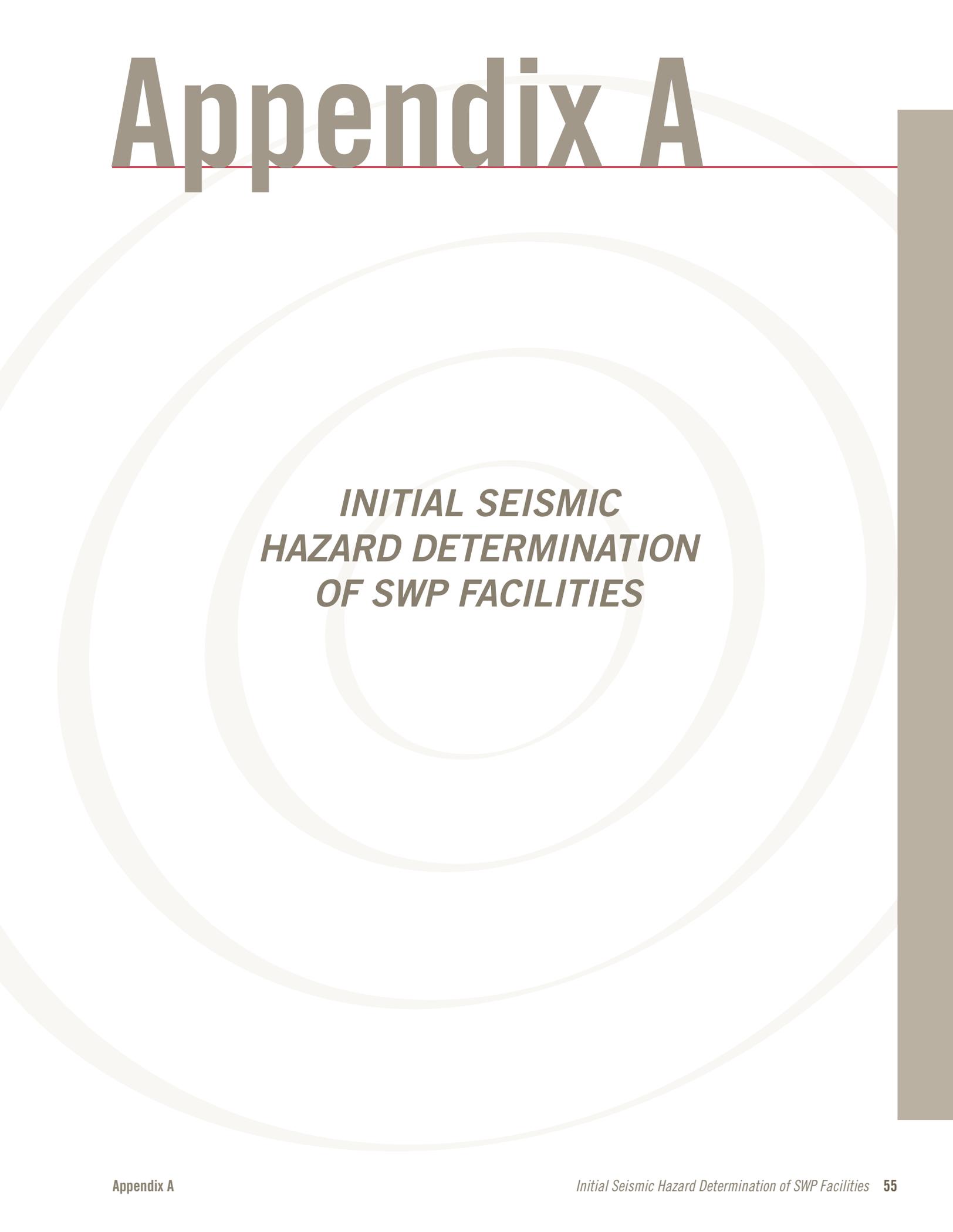
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Appendix A



***INITIAL SEISMIC
HAZARD DETERMINATION
OF SWP FACILITIES***

1.0 INTRODUCTION

Much of California, and hence most of the region traversed by the State Water Project (SWP) is seismically active. The northern portion of the SWP in the Upper Feather River and Lake Oroville area is relatively seismically quiet but, through the San Joaquin Valley and into Southern California, the conveyance system roughly parallels the San Andreas fault, the source of many large earthquakes. The California Aqueduct crosses the San Andreas fault at four places: Quail Lake, Anaverde Valley, Barrel Springs near Palmdale, and at Devil Canyon Powerplant. Other major fault crossings are the Garlock fault zone in the Tehachapi Mountains and the San Jacinto fault south of the San Bernardino Mountains. The West Branch also crosses the San Andreas fault, and the South Bay Aqueduct crosses the Calaveras fault. In addition to these major faults, numerous minor faults are crossed by various features of the Project. Furthermore, a number of other mapped faults terminate adjacent to the aqueduct and may or may not completely cross it. At other locations faults trend along-side and parallel to the aqueduct. And at still other locations, blind thrust faults that do not rupture to the surface underlie the SWP. All of these faults pose some level of hazard to the SWP.

The proximity to major active fault zones poses the hazard of damage by ground shaking from large earthquakes occurring along these zones. Ground shaking potential (ground motions) has been calculated for 18 facilities along the length of the SWP (See Figure A.1 for a location map of the facilities). These facilities generally consist of Pumping Plants (PP); however the Hyatt Power Plant, Napa Turnout and South Bay Aqueduct Terminal reservoirs were also included in this ground motion study. The study was conducted in order to establish an understanding of the various levels of ground motion to which the SWP may be subjected during a nearby earthquake event. Comparison of these various levels of potential ground motion to the current design level of the facilities will ultimately be used to establish the minimum seismic loading criteria for all SWP facilities. It is important to note that these ground motions should not be used for design primarily because the 18 facilities studied are generally founded within deep excavations and thus possess better foundation conditions than adjacent appurtenant structures such as support buildings and the aqueduct. Furthermore, even though every attempt was made to properly characterize the foundation conditions at the facilities, the foundation conditions are assumed and should be verified.

The potential for displacement of the SWP exists at fault crossings as well as where the SWP is located very near active faults. In the ensuing discussion, we consider only faults that could rupture and displace the ground surface during large magnitude earthquakes.

In this appendix, ground motions will be covered first followed by a discussion on hazards pertaining to fault rupture.

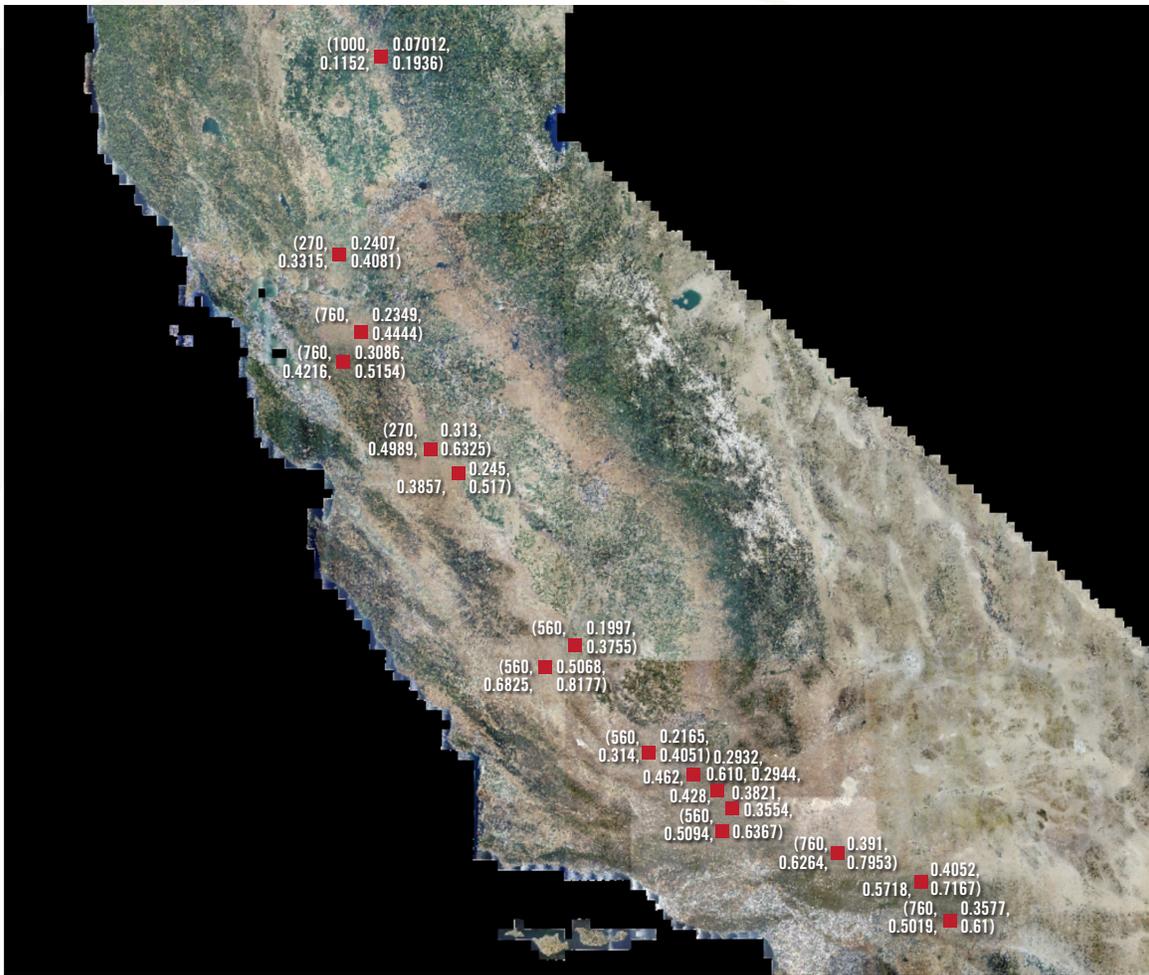


Figure A.1 Location Map of 18 locations where ground motions were calculated

2.0 GROUND MOTIONS

The 18 sites were primarily selected to provide a representative sampling along the SWP. However, a site's location relative to faulting also was considered and when possible, sites closer to faults were chosen. The ground motions were determined using EZ-FRISK. The EZ-FRISK program calculates seismic hazard using both probabilistic (PSHA) and deterministic (DSHA) procedures based upon user selected attenuation equations and seismic sources.

Both probabilistic (return periods from 200 to 3000 years) and deterministic (median and 84th percentile) ground motion estimations have been performed using the Uniform California Earthquake Rupture Forecast, Version 2 (UCERF2) as the fault model and the five Pacific Earthquake Engineering Research (PEER) Center's Next Generation Attenuation (NGA) relationships. Additionally, current California Building Code (CBC 2010) design spectra (ASCE 7-05) and the proposed future International Building Code (IBC 2012) design spectra (ASCE 7-10) have been determined for each site for comparative purposes.

2.1 NEXT GENERATION ATTENUATION RELATIONSHIPS

The NGA relationships are the product of a multidisciplinary research program coordinated by PEER with the objective of developing new ground motion attenuation relations (ground motion prediction equations, or GMPEs) for shallow crustal earthquakes in the western United States and similar active tectonic regions.

The NGA models improve upon earlier GMPEs in that they:

- predict ground motion parameters of peak ground acceleration (PGA), peak ground velocity (PGV), and five percent damped elastic pseudo-response spectral accelerations throughout a greater period range (0 – 10 seconds);
- predict
- are applicable to a greater moment magnitude range (Mw 5 to 8.5 for strike-slip earthquakes and Mw 5 to 8 for reverse and normal earthquakes); and
- are applicable to distances up to 200 km.

In comparison, the older relations were able to predict response spectral values to periods varying from zero to five seconds, PGV was not addressed in many relationships, the largest applicable magnitudes varied from 7.5 to 8, and distances of applicability were limited to 100 km. The ground motions were determined using the arithmetic mean of all five NGA models with the exception of several sites where the estimated V_{s30} was lower than the 450 m/s, which is the minimum allowed in the Idriss relationship and therefore, the Idriss model was not used at those sites.

2.2 SEISMIC SOURCE CHARACTERIZATION

Seismic hazard analysis begins with an earthquake rupture forecast – a model of probabilities that earthquakes of specified magnitudes, locations, and faulting types will occur during a specified time interval.

The type of earthquake source (faults and background earthquakes), their geometry and recurrence intervals used in this analysis are described in UCERF2. UCERF2 constitutes the latest and most up to date earthquake rupture forecast for California developed by the 2007 Working Group on California Earthquake Probabilities (WGCEP 2007).

2.3 SITE CONDITION (Vs30)

The NGA relationships primarily characterize site condition by the average shear wave velocity within the upper 30 meters of the site (Vs30). Three of the NGA models also require an additional “soil” depth term in the form of Z1.0 (depth to Vs = 1 km/sec - Abrahamson and Silva, and Chiou and Youngs) or Z2.5 (depth to Vs = 2.5 km/sec – Campbell and Bozorgnia). In the absence of measured data, the NGA models provide a calculation for Z1.0 and Z2.5 based upon the Vs30; that option was used in this analysis.

Upon review of subsurface information contained in various project design and construction reports housed in the Division of Engineering’s (DOE) Project Geology Section (PG) library, staff from DOE’s, Dams and Canals Section made the initial Vs30 estimates for each site. The data in the reports was primarily used to establish a foundation material type (soil or rock) and general density state (soft to stiff, etc.), and geological formation. The data was then compared to data available from the California Geological Survey (CGS) that relates site geology to the National Earthquake Hazards Reduction Program (NEHRP) site class designations. While no measured Vs30 values were observed in the project files, many of the files contain detailed boring log information as well as CPT logs, seismic refraction testing and laboratory testing.

The (currently unpublished) CGS database is largely based upon the work of Wills and Clahan (2006) where they sorted available shear-wave velocity data by geologic unit, generalized the geologic units, and prepared a map which was then used to transfer the velocity characteristics from the sites where they were measured to sites on the same or similar geologic material. An example of how the Project Geology file data and CGS data were compared is described below. Suppose the Project Geology files indicate that a site is located within Panoche formation but more specifically that the silt and claystones of the Panoche formation at this site are moderately weathered. The site class indicated for the Panoche formation in the CGS database at this site is Site Class C which ranges from 365 m/s to 760 m/s. Because of the moderately weathered description noted in the Project Geology file an average Vs30 in the Site Class C range was chosen--approximately 560 m/s.

In the absence of in-situ measurements, Vs30 for soil and very soft to soft rock can be estimated using several correlations. For cohesive soils, correlations with laboratory measured undrained shear strength are available. For cohesionless soils, correlations with either SPT blow count or CPT tip resistance are available. For harder rock, the mass shear wave velocity may be evaluated based

upon correlations with other engineering and physical properties of rock mass and rock cores measured in the field or laboratory, or estimated using geologic correlations such as the CGS database. The boring logs did not generally contain sufficient data to utilize these correlations.

Initial Vs30 estimates were produced for each facility using the boring data and CGS database as described above and were then provided to Project Geology staff for their review and input. Based upon their experience with the facilities, staff prepared a table with Vs30 estimate ranges for each facility. In most cases, Dams and Canals' Vs30 estimate fell within Project Geology's estimated range. When this occurred, Dams and Canals' Vs30 estimate was used in the analysis. In several cases, the Dams and Canals' estimate fell outside of the Project Geology's estimated range; the differences were usually 200 m/s or less. To evaluate the impact of varying Vs30 on the individual site spectra, a sensitivity analysis was conducted using EZFrisk and varying the Vs30 over a range of values representing the estimates. The sensitivity analysis indicated that changes in Vs30 of 200 m/s or less had minor impact on the spectra. Therefore, in the cases where the Dams and Canals' estimate was outside of the Project Geology's estimated range, the Dams and Canals' estimate was adjusted by up to 200 m/s to bring it within or closer to the Project Geology estimate range. Finally, when not much information is available upon which to base a Vs30 estimate at a site, a more conservative (lower) Vs30 value was selected from either the D&C or PG estimates. It should be noted that no in-situ Vs30 data is currently available for the SWP sites included in this study. The following Table presents the Vs30 estimates that were used in the development of ground motions.

Table A.1 Vs30 estimates used in initial ground motion development

Facility	Report Referenced Project Geology Report No.	CGS Mapped Vs30 Estimate (NEHRP class)	Dams and Canals Vs30 Estimate	Project Geology Vs30 Estimate Range	Final Vs ₃₀ Estimate
Hyatt Power Plant	20-11-03, 20-21-01	1000 m/s (B)	1000 m/s	1000 – 1200 m/s	1000 m/s
Barker Slough Pumping Plant	30-00-21	360 m/s (C/D)	270 m/s	560 - 760 m/s	270 m/s
Napa Turnout Reservoir	30-00-23, C-145	560 m/s (C)	560 m/s	560 - 760 m/s	560 m/s
Del Valle Pumping Plant	D-9, D-61, 40-30-08, C-44	760 m/s (B/C)	560 m/s	760 - 900 m/s	760 m/s
SBA Terminal Tank	40-11-09, C-12, D-30	270 m/s (D)	360 m/s	360 m/s	360 m/s
Banks Pumping Plant	51-31-13, 51-31-23, C-54, 51-10-16	1000 m/s (B)	760 m/s	760 – 900 m/s	760 m/s
Gianelli Pumping/Generating Plant	52-00-36	270 m/s (D)	270 m/s	270 m/s	270 m/s
Dos Amigos Pumping Plant	C-153	360 m/s (C/D)	360 m/s	270 – 760 m/s	360 m/s
Las Perillas Pumping Plant	D-74, 55-00-04	360 m/s (C/D)	560 m/s	560 - 760 m/s	560 m/s
Polonio Pass Pumping Plant	55-02-26, D-146, C-111	360 m/s (C/D)	560 m/s	560 - 900 m/s	560 m/s
Buena Vista Pumping Plant	D-89, 53-30-05	560 m/s (C)	560 m/s	560 - 760 m/s	560 m/s
Teerink Pumping Plant	D-95, 53-30-12	560 m/s (C)	360 m/s	560 - 900 m/s	560 m/s
Edmonston Pumping Plant	54-31-02	1000 m/s (B)	760 m/s	560 – 900 m/s	760 m/s
Oso Pumping Plant	D-66, 56-31-01, 56-31-02	360 m/s (C/D)	360 m/s	560 - 1000 m/s	560 m/s
Warne Power Plant	D-130, 56-20-06, 56-20-02	360 m/s (C/D)	560 m/s	560 – 900 m/s	560 m/s
Pearblossom Pumping Plant	57-31-01, 57-31-08, D-58, C-48	360 m/s (C/D)	560 m/s	760 - 1100 m/s	760 m/s
Mojave Siphon Power Plant	D-140, C-104	760 m/s (B/C)	760 m/s	760 - 1100 m/s	760 m/s
Cherry Valley Pumping Plant	D-156	560 (C)	560 m/s	760 - 900 m/s	760 m/s

As indicated in the table above, most of the ground motion estimates were derived at pumping and power plant sites. Generally these facilities are founded upon firm materials at the base of substantial excavations. Therefore, using these ground motions for evaluation of adjacent facilities founded at or near the existing ground surface (e.g. administration buildings, aqueduct, etc.) is only warranted if similar foundation conditions exist.

2.4 FAULT RUPTURE DIRECTIVITY

Average fault rupture directivity was included in the estimated ground motions using the model by Somerville et al. (1997) as modified by Abrahamson (2000). EZFrisk applies the Summerville directivity procedure in the DSHA by conservatively locating the hypocenter such that maximum effect is calculated. For the PSHA, EZFrisk randomly locates the hypocenter along the length of the fault to account for the lack of a priori knowledge of the rupture initiation.

2.5 ESTIMATED RESPONSE SPECTRA

Spectral plots presenting the probabilistic (200-, 500-, 1000-, and 3000-year return period) and deterministic (50th and 84th percentile) response spectra as well as the current CBC (ASCE 7-05) and proposed future IBC (ASCE 7-10) design response spectra for the selected sites are in Figures A 5 through A 20. All probabilistic and deterministic spectra were determined using EZFrisk, NGA, UCERF2 and the Vs30 estimates except for the deterministic results for Hyatt Power Plant which were based upon the Thermalito Diversion Dam Faulting and Seismicity Report (Project Geology Report No. 20-13-41) findings that suggests the Swain Ravine fault (not included in UCERF2) is the controlling fault in the area.

The figures indicate that in general the maximum spectral response of the 18 chosen facilities is in the range of 0.2 to 0.4 seconds. The 84th percentile deterministic ground motions are generally between 1000- to 3000-year return period, with the exception of Del Valle PP, SBA Terminal Reservoir, and Dos Amigos PP which have 84th percentile ground motions that are less than 1000-year return period and Hyatt Power Plant and Teerink PP which have 84th percentile motions that are greater than 3000-year return period. The median deterministic ground motions for the 18 facilities are generally between 200- and 500-year return period motions except for Del Valle PP and SBA Terminal Reservoir which have median ground motions that are less than 200 year return period and Hyatt Power Plant, Teerink PP and Warne Power Plant which have higher than 500 year return period median ground motions.

The figures generally indicate that the minimum seismic design level of the future revision of the building code (ASCE 7-10) will be equal to or higher than the current building code requirements (ASCE 7-05) with the exception of Barker Slough, Dos Amigos, Teerink and Edmonston PPs which will likely encounter lower minimum seismic design levels under the new code. Comparing the building code minimum seismic design levels to the probabilistic ground motions indicates the building code minimums are approximately equal to the 1000-year return period ground motions for the Banks, Gianelli, Dos Amigos, Las Parillas, Buena Vista, Teerink and Edmonston PP facilities, and are greater than 1000-year return period motions for Hyatt Power Plant and Barker Slough PP. The code minimums are lower than 1000-year return period levels for the NBA Terminal; Del Valle, Oso, Cherry Valley, and Pearblossom PPs; Warne and Mojave Siphon Power Plant facilities, and are lower than 500-year return period motions at the SBA Terminal and Polonio Pass PP facilities. Recommended minimum spectra for SWP facilities (power plants, pumping plants, canals, dams, etc.) are presented in the specific chapters pertaining to these facilities.

3.0 *SURFACE FAULT RUPTURE*

As previously stated, there are approximately 67 locations along the SWP where active faults cross the aqueduct. In the previous discussion, we considered the potential shaking that a site could experience resulting from an earthquake event occurring some distance from the site. In the following discussion, we consider what the ground surface displacement could be along a major fault trace during large magnitude earthquakes.

Primary fault ruptures occur on the main continuous fault that is located within several meters of the mapped fault trace resulting in principal surface fault displacements. The term principal faulting refers to coseismic surface rupture that occurs along the fault or faults responsible for the release of seismic energy during an earthquake (Coppersmith and Youngs, 2000).

Distributed ruptures can occur from tens of meters to many kilometers away from the principal fault trace, are often discontinuous, and can occur on a variety of structures either related to the principal fault that ruptures during an earthquake, or along separate structures with no direct connection to the principal fault either at the surface or at depth. Such structures include parallel to sub-parallel faults, splay faults that branch away from the principal fault trace, as well as regional faults that are structurally unrelated to the principal fault that produced the earthquake. Distributed ruptures and their associated displacements can be along faults and structures that are unrecognized or are too small to be considered an independent seismogenic source (Peterson and others, 2011).

Surface rupture displacement on principal faults is generally largest near the middle of the fault and may fall off rapidly along the length towards the ends of the rupture (Hemphill-Haley and Weldon, 1999, Peterson and others, 2011). However, some earthquakes have their highest displacements near the end of the rupture (e.g., 1968 Borrego Mountain, Wesnousky, 2008). Figure A.2 is a comparison of average fault displacements along principal faults versus earthquake magnitude.

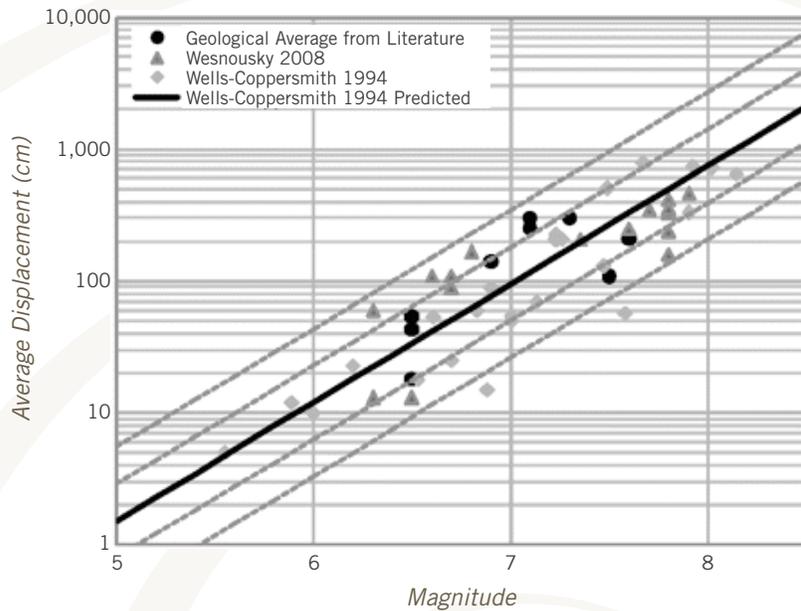


Figure A.2

Comparison of the average displacement data from geologic literature and Wesnousky (2008) with those of Wells and Coppersmith (1994). All data fall within the two standard deviations of the Wells and Coppersmith (1994) regression.

Peterson and others (2011) have determined that one of the most important results from their analysis indicates that, while observed ground displacements on the principal faults are generally quite large (meters), the displacements measured off the fault are generally only a few percent of the principal fault displacements. Secondary ruptures have been observed to exceed one meter. However, these centimeter scale displacements may occur several kilometers from the primary fault, on distributed immature faults.

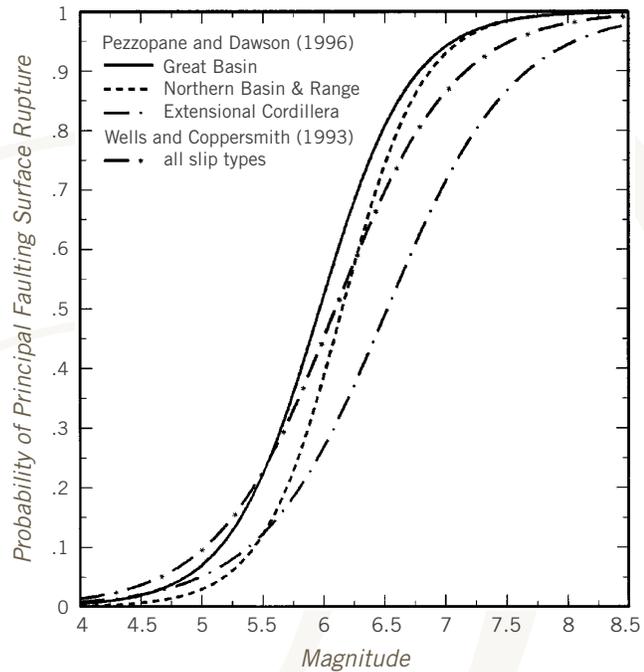
If an engineering project is located in the vicinity of a known fault, but insensitive to centimeter levels of displacement, then it is essential that an engineering geologist verifies that the site is not located on the observed main strand or unidentified (unmapped) faults located nearby. If the site is sensitive to centimeter-size displacements, then the engineer may need to design for fault rupture even if the site is located a few kilometers from the known earthquake fault source.

Earthquake surface fault rupture is dependent on the dynamic environment of the fault. The rapid release of stress built up along the fault surface (stress drop), the fault rupture length, and the fault width from surface to depth are the components that make up the dynamic environment. Although statistics show that surface fault displacement appears in most instances at magnitudes of about 6.1 (~M6.1), data from certain regions indicate that seismicity at superficial depths is, under certain conditions, accompanied by surface fault displacement even for magnitudes as small as M5.5 (Figure A.2). The threshold magnitude for surface faulting mostly depends on the rheology of rock in the fault area and on the stress environment (Mohammadioun and Serva, 2001).

The probability of surface fault rupture because of earthquakes was demonstrated graphically by Pezzopane and Dawson (1996) and Wells and Coppersmith (1993) (Figure 2). Although beyond the scope of this report, the various authors used statistics to support their probabilistic estimates

demonstrated in Figure A.3. Generally, there is a 50 percent chance of surface rupture from a magnitude earthquake of about M6; an 80 percent chance of surface rupture from a magnitude earthquake of about M6.5; and a 95 percent chance of a surface rupture from a magnitude earthquake of about M7. However, these probabilities do not estimate the amount of surface displacement experienced during or after the earthquake.

Figure A.3
Probability of surface fault rupture from earthquakes along principal faults.



For detailed methodologies of estimating the amount of surface displacement resulting from earthquake faulting using probabilistic and deterministic analysis, see Youngs and many others (2003), and Peterson and others (2011), respectively. The details of the methodologies described in these papers are beyond the scope of this report.

3.1 METHOD OF FAULT DISPLACEMENT ESTIMATION

Earthquakes are typically the result of a rapid release of energy stored in the earth's crust along fault planes. The released energy radiates in all directions from the source location in the crust called the focus, or hypocenter. Strong earthquakes result from the sudden slip of the crust at the hypocenter. This hypocenter slip is on the order of about one meter of displacement (~M7.0) along faults that may, or may not, rupture to the earth's surface. Large earthquakes may rupture the ground surface laterally, vertically, or in some combination of both vertical and horizontal directions. Large earthquakes may not always rupture to the ground surface, as in the case of blind thrust faults, but instead deform (warp) the surface by raising or lowering the regional topography. Regardless of the ground surface's displacement or deformation, large earthquakes will likely have some impact on the SWP in the future.

In preparation of future large earthquakes, DWR engineers must know the location of surface fault traces and to what degree surface fault rupture will occur. In addition to surface fault rupture along the SWP, engineers would also like to know the degree to which surface deformation may occur because of earthquakes that do not rupture to the surface, but instead deform the topography enough to affect the flow of water through the SWP. Unfortunately, surface deformation estimates are difficult to determine because of the variability of the earth's crust at depth and the crust's response to subsurface displacement. Therefore, this section will only address surface fault rupture displacement estimates and not surface deformation (warping) estimates.

3.1.1 Fundamental Seismology Mechanics

In order to estimate the amount of displacement at the surface of the earth because of a large earthquake, several fundamental seismologic factors need to be explained.

The most commonly used earthquake magnitude scale used today is the moment magnitude (M_w) scale, jointly developed in 1978 by Dr. Thomas C. Hanks of the USGS and Dr. Hiroo Kanamori, a professor at CalTech. Moment magnitude is related to the physical size of fault rupture and the movement (displacement) across the fault. The moment magnitude (M_w) definition is written:

$$M_w = 2/3 \log M_o - 10.7 \quad (1)$$

where 'w' represents work done.

In order to obtain the moment magnitude, we must estimate the seismic moment (M_o) first. The seismic moment is typically measured in dyne-centimeters (dyne-cm). The seismic moment of an earthquake is determined by the strength or resistance of rocks to faulting. The strength or resistance of the rock is the shear modulus "u", typically measured in dyn-cm². The shear modulus is then multiplied by the area "A" of the fault plane that ruptures. The fault plane is typically measured in cm². The area is then multiplied by the average displacement "D" of the fault plane area ruptured, typically measured in centimeters (Figure A.4). The seismic moment definition is written:

$$M_o = u A D \quad (2)$$

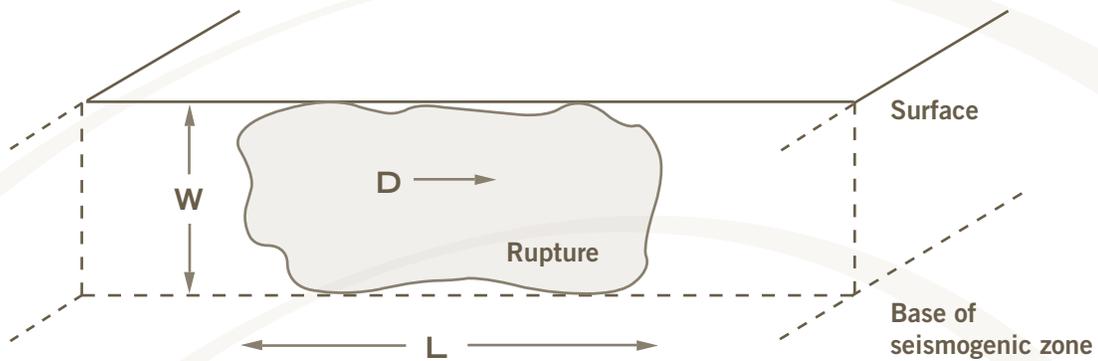


Figure A.4

Seismic moment ($M_0 = \mu AD$) is equal to the shear modulus (μ) of the crust (about 30 gigapascals, ~ 30 GPa) in the crust, multiplied by the area (A) of the rupture (length times width), multiplied by the average displacement (D) during the rupture. The maximum area of rupture is controlled by the depth to the base of the seismogenic zone to the surface of the earth's crust. A typical depth used in calculating area of rupture in California is about 15 kilometers of fault width, depending on region.

The seismic moment determines the energy that an earthquake can radiate. A seismologist typically determines the seismic moment of an earthquake from a seismogram by using a computer to plot the seismogram's amplitude of motion as a function of period (wave length). The amplitude of the long period motions in a seismogram, when corrected for the distance from the earthquake, is a measure of the seismic moment for that earthquake.

The moment magnitude of an earthquake is defined relative to the seismic moment for that event. It is important to recognize that earthquake magnitude varies logarithmically with the wave amplitude or seismic moment recorded by a seismograph. Each whole number step in magnitude represents an increase of ten times in the amplitude of the recorded seismic waves and the energy release increases by a factor of about 32 times. An increase of two steps corresponds to approximately 1,000 times an increase in energy. The size of the fault rupture and the fault's displacement also increase logarithmically with magnitude. Magnitude scales have no fixed maximum or minimum.

3.1.2 Displacement Estimate Method

Equations relating moment magnitude of an earthquake to the surface fault displacement are poorly constrained. Most recently, Peterson and others (2011) have prepared methodology for evaluating the hazard of fault displacement in a probabilistic and deterministic framework.

Because of variability in determining surface fault displacement estimates, we recommend using regressions of the Wells and Coppersmith (1994) maximum surface displacement estimates from maximum magnitude earthquakes. Maximum ground surface displacement (MD) estimates are based on regression analysis of Wells and Coppersmith 1994 equation (3):

$$M = 6.69 + 0.74 \cdot \log(\text{MD}) \quad (3)$$

Where, “M” is magnitude and “MD” is expressed in meters. Equation 3 considers all fault types (strike-slip, reverse, and normal). It is important to note that the coefficients change in equation (3) when solving for magnitude for individual fault types (strike-slip, reverse, and normal). Maximum magnitude earthquakes on known faults are primarily determined from the UCERF2, 2008 California fault database, prepared by the 2007 WGCEP.

Equation (4) located below can effectively determine the maximum ground surface displacement (MD).

$$\text{Log (MD)} = -5.46 + 0.82 * M, \quad (4)$$

Note that the coefficients in equation (4) are not the same as in equation (3). Because of deviations and correlation variations of the data sets Wells and Coppersmith (1994) used in their analysis, the regression equations (equations (3) and (4)) for surface fault displacement and moment magnitude are not similar.

3.1.3 Example Calculation for North Bay Aqueduct, Mile Post 22.69, 22.80, and 23.01

The following example is an estimate of a first-order approximation of surface displacement for principal faults potentially affecting the SWP. In this example we do not consider the likelihood or probability of a fault rupture event, just that the potential exists. The surface displacement calculated in this example is the offset that would be expected from a maximum magnitude earthquake event.

The USGS Quaternary Fault and Fold Database shows the eastern branch of the Green Valley fault zone as both moderately and well constrained at these locations. The Green Valley fault zone crosses the North Bay Aqueduct pipeline at about Mile Post 22.69, about 3,000 feet upstream (east) of the Cordelia Surge Tank. The central branch of the Green Valley fault zone is well constrained and crosses the North Bay Aqueduct pipeline at about Mile Post 22.80, about 2,500 feet upstream (east) of the Cordelia Surge Tank. The western branch of the Green Valley fault zone is well constrained and crosses the North Bay Aqueduct pipeline at about Mile Post 23.01, about 1,400 feet upstream (east) of the Cordelia Surge Tank.

Because of the close proximity of the faults to one another along the pipeline (about 700 feet), we consider the fault geometries identical for surface fault rupture displacement estimates.

The Green Valley fault zone is located on the USGS 7.5 Minute Topographic Map for the Cordelia Quadrangle. The western and central branches of the Green Valley fault zone (along the pipeline) are included in the Alquist-Priolo Earthquake Fault Zone maps; the eastern branch of the fault is not included in the fault zone map. There is reliable information available on the Green Valley fault zone in this area from the USGS fault database, compiled from the CGS.

We use fault geometry data from UCERF2 for estimating surface fault rupture displacement for the three branches of the Green Valley fault zone. Table 2 lists fault geometry parameters taken from UCERF2 for the Green Valley fault zone.

Table A.2

Green Valley fault zone geometry parameters and Mile Post crossing location.

Fault Name	Green Valley fault zone (three branches)
Fault Style	Right lateral strike slip
Fault Dip	90 degrees
Fault Length	56.4 km
Fault Plane Width	14 km
Maximum Magnitude (moment magnitude)	Mw 6.9
Approximate Displacement	1.9 meters
Approximate Mile Post Location Crossing	MP 22.69 , MP 22.80, and MP 23.01

For this surface fault rupture displacement example, we used the fault parameters listed in Table 1 and Ross Stein's 2008 Magnitude-Area Relationship (UCERF2, 2008) (equation 5) to approximate the earthquake size:

$$M_w = 4.2775A^{0.0726}, \quad (5)$$

where "A" is area of fault plane rupture measured in square kilometers (km²).

Using the above information, we have estimated the maximum magnitude earthquake on the Green Valley fault zone to be approximately M 6.9. Using regressions of Wells and Coppersmith (1994) equations, we estimate the approximate surface fault rupture displacement near MP 22.69, MP 22.80 and MP 23.01 to be approximately 1.9 meters.

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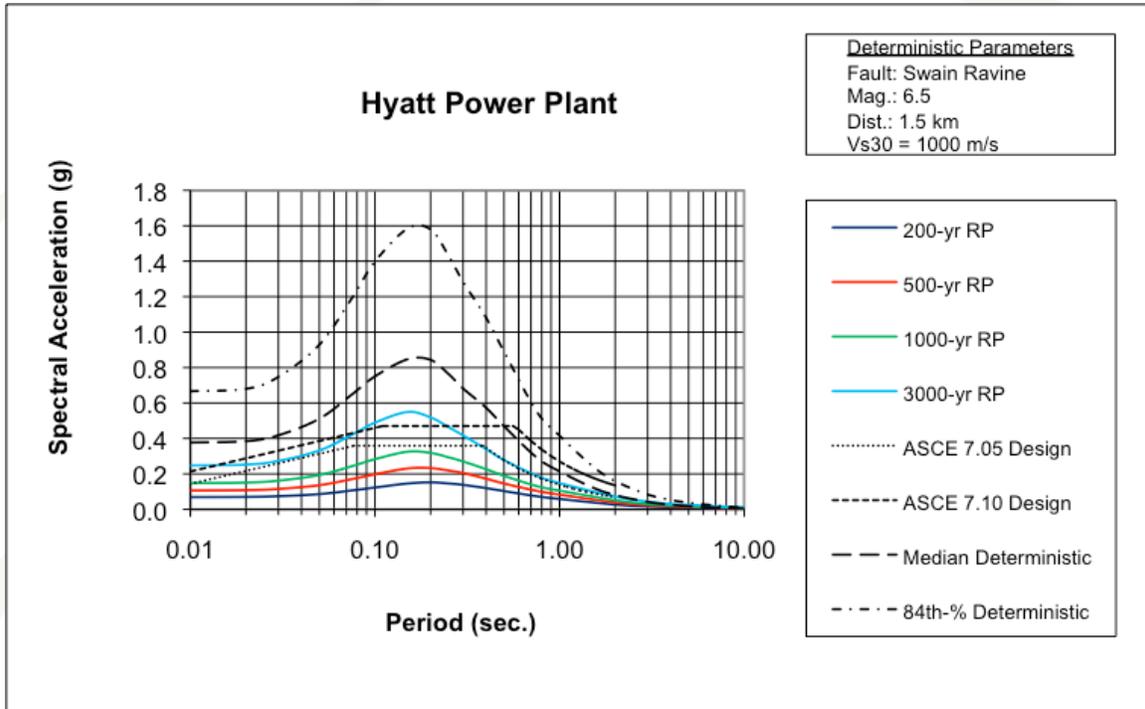


Figure A.5
 Hyatt Power Plant Estimated Ground Motions

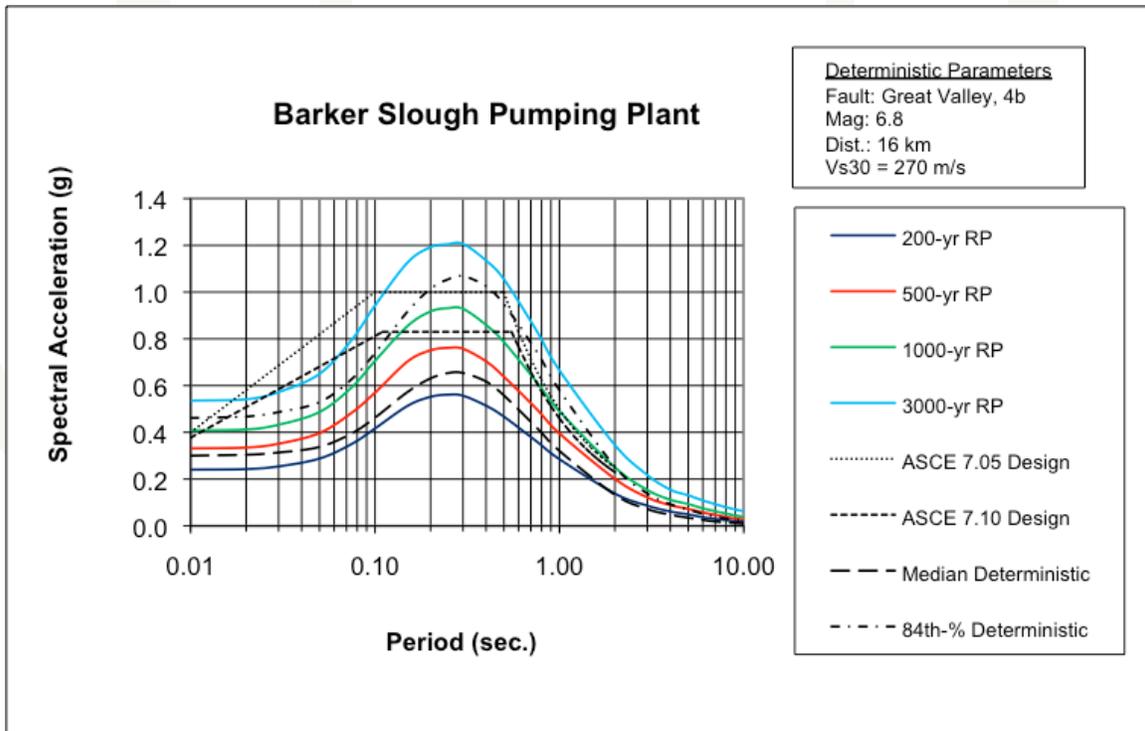


Figure A.6
 Barker Slough Pumping Plant Estimated Ground Motions

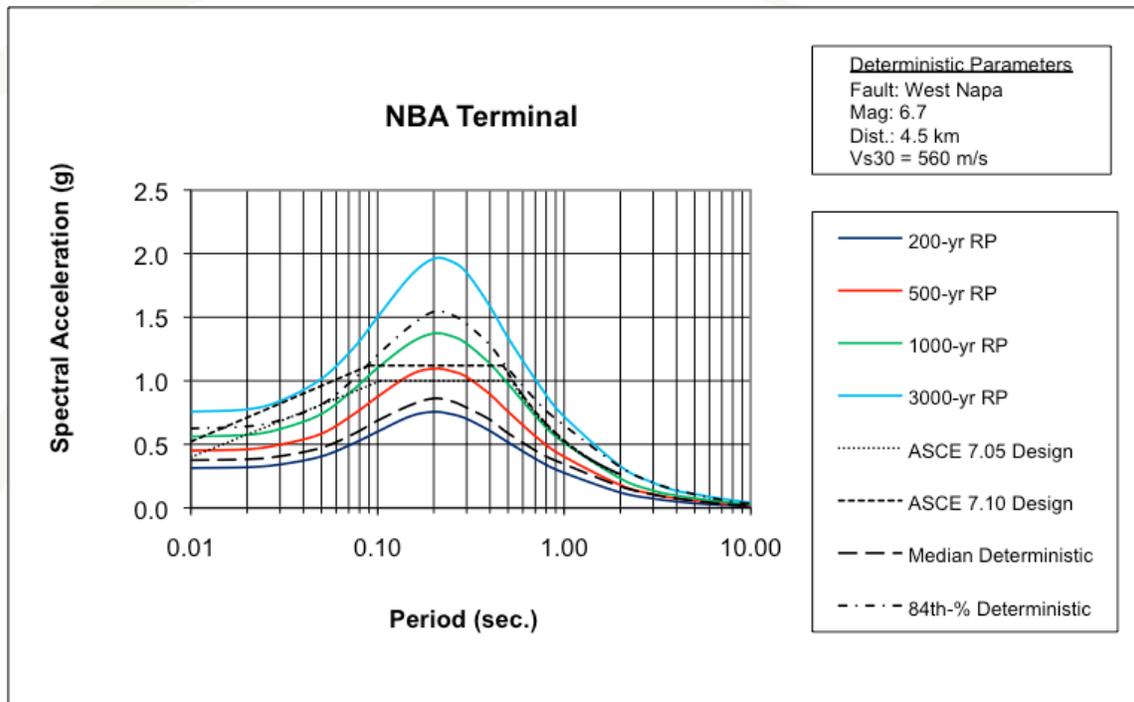


Figure A.7
 NBA Terminal Tank Estimated Ground Motions

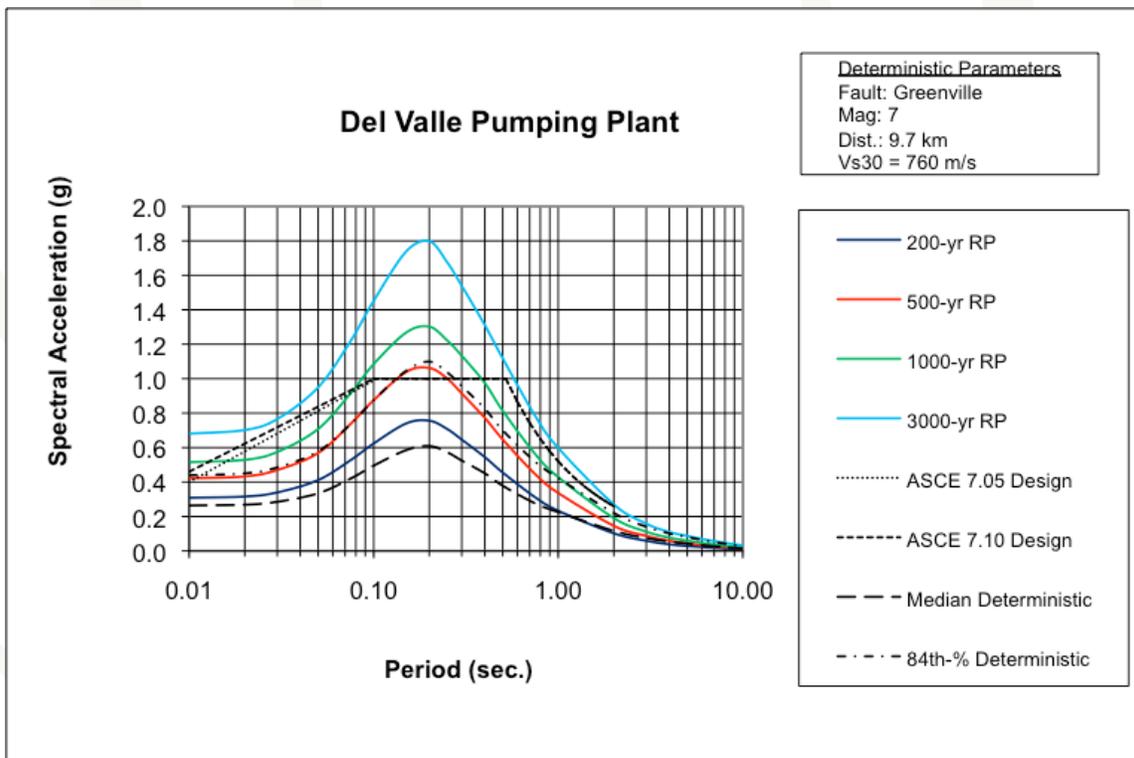


Figure A.8
 Del Valle Pumping Plant Estimated Ground Motions

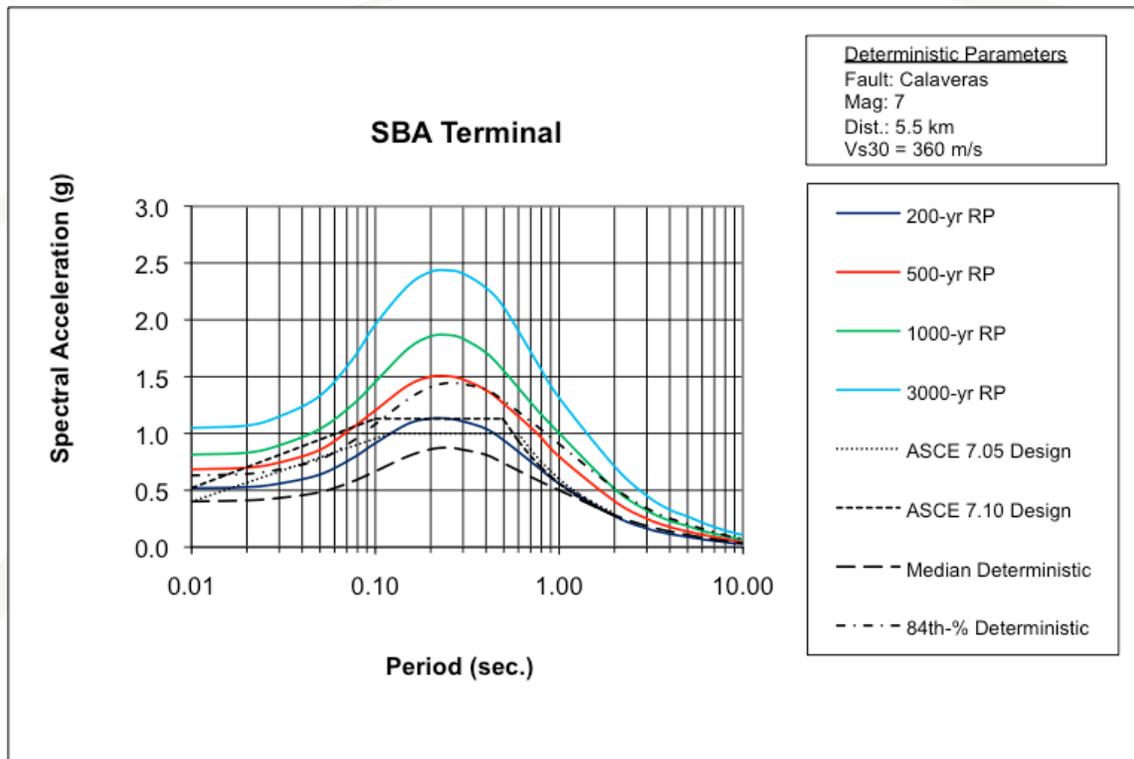


Figure A.9
 SBA Terminal Tank Estimated Ground Motions

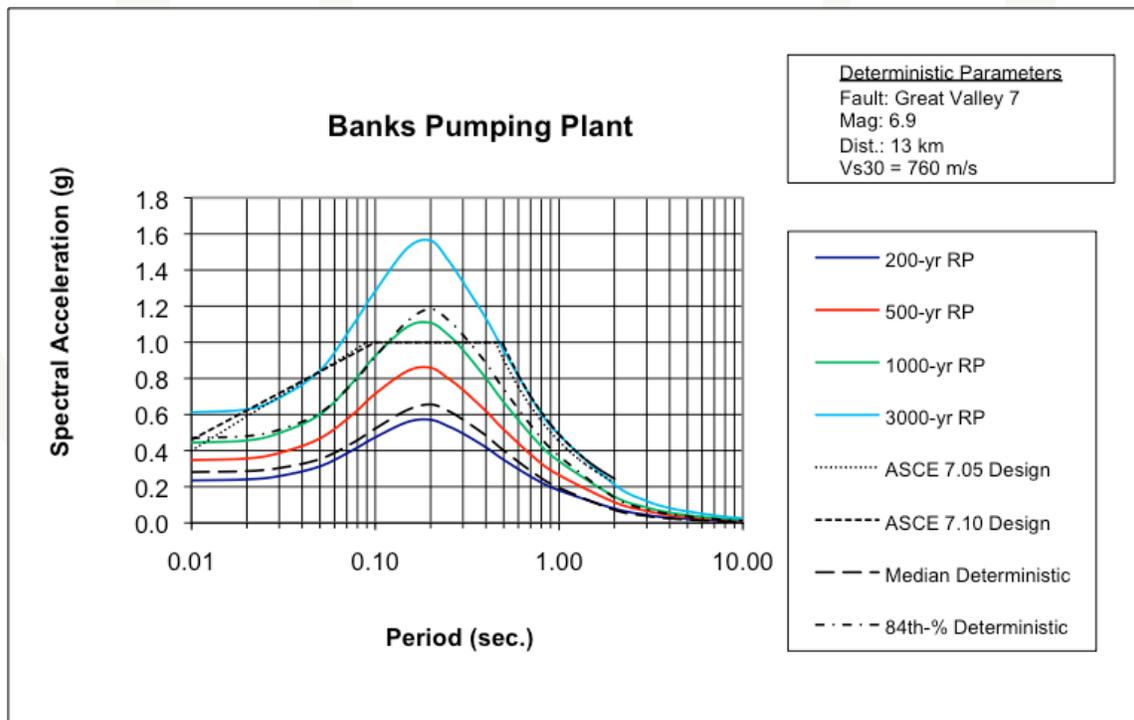


Figure A.10
 Banks Pumping Plant Estimated Ground Motions

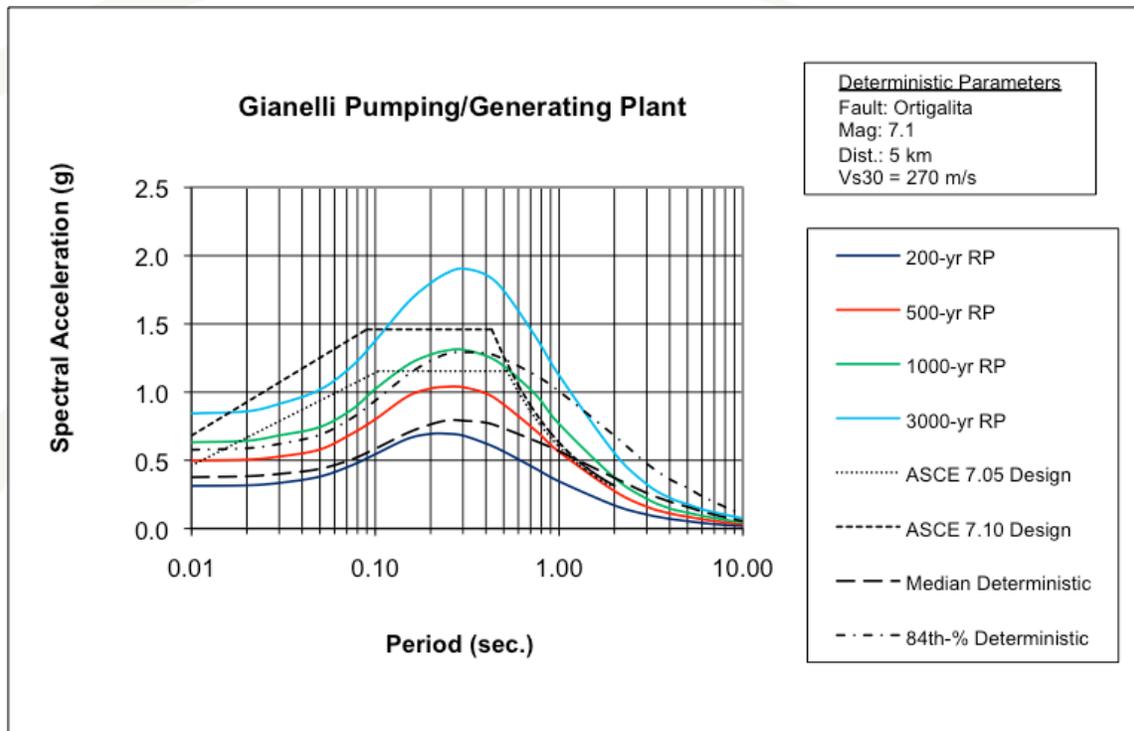


Figure A.11
 Gianelli Pumping/Generating Plant Estimated Ground Motions

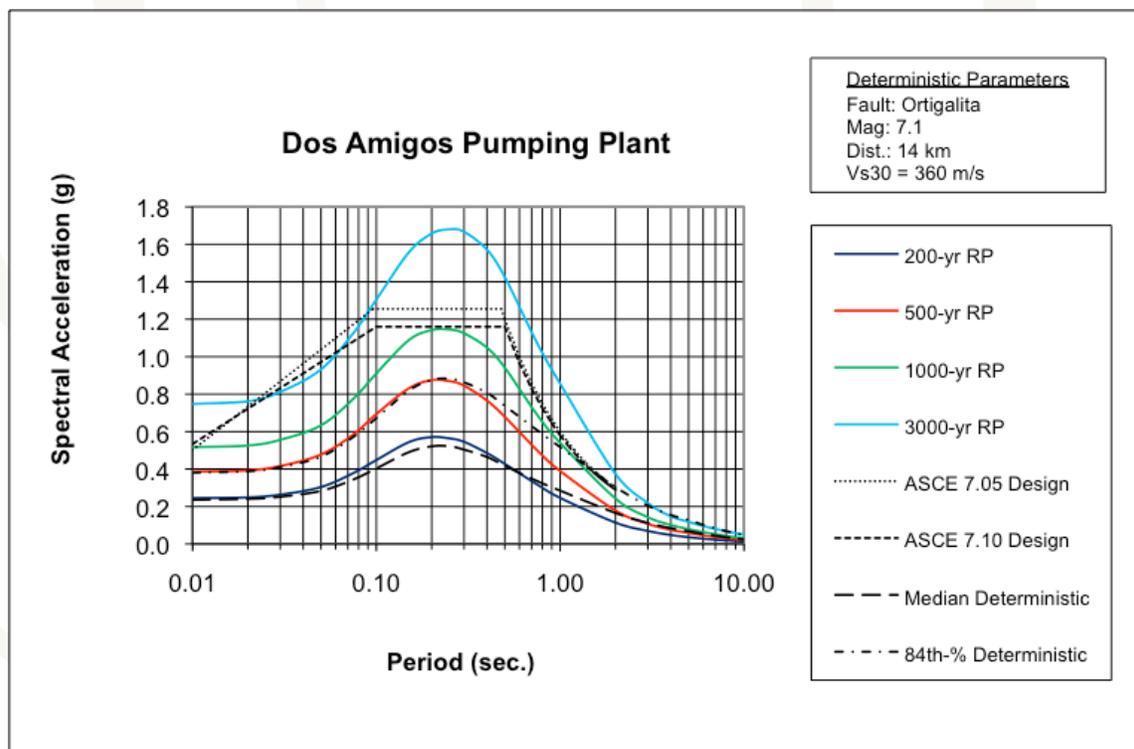


Figure A.12
 Dos Amigos Pumping Plant Estimated Ground Motions

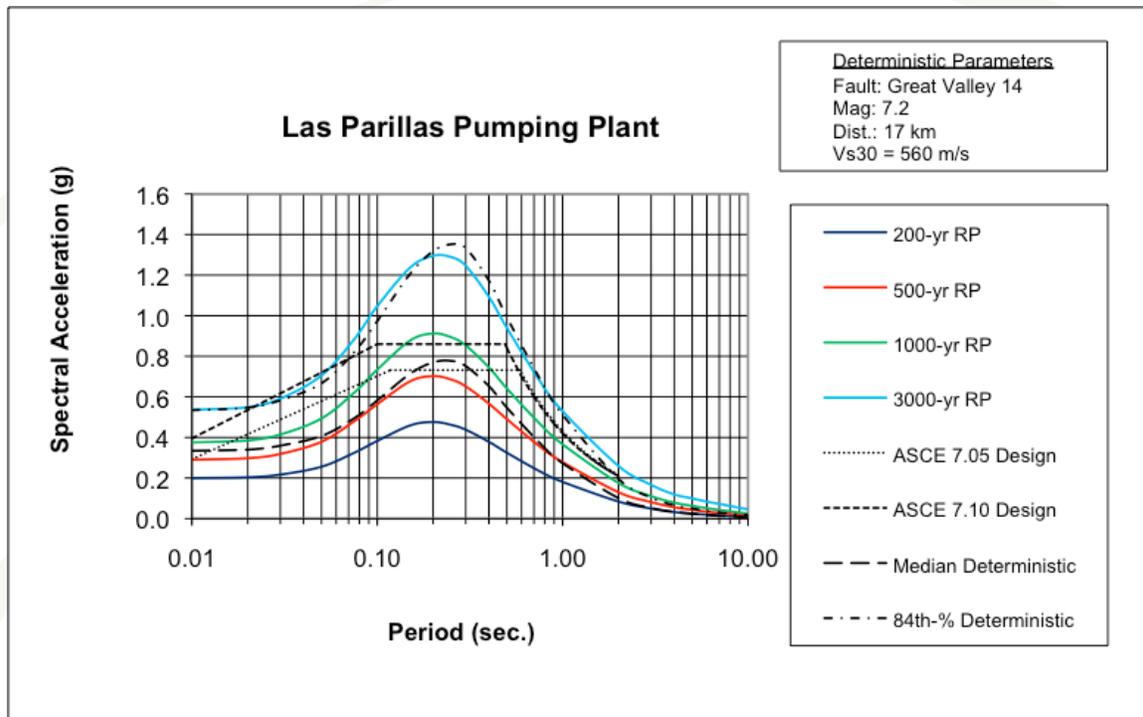


Figure A.13
 Las Parillas Pumping Plant Estimated Ground Motions

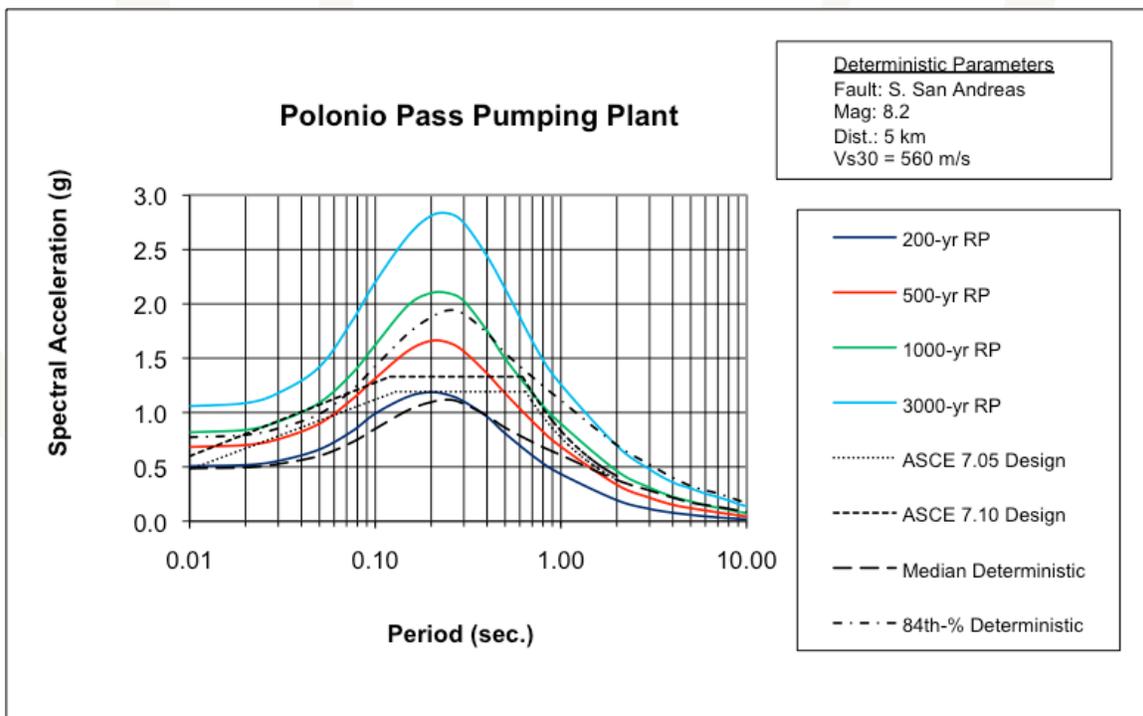


Figure A.14
 Polonio Pass Pumping Plant Estimated Ground Motions

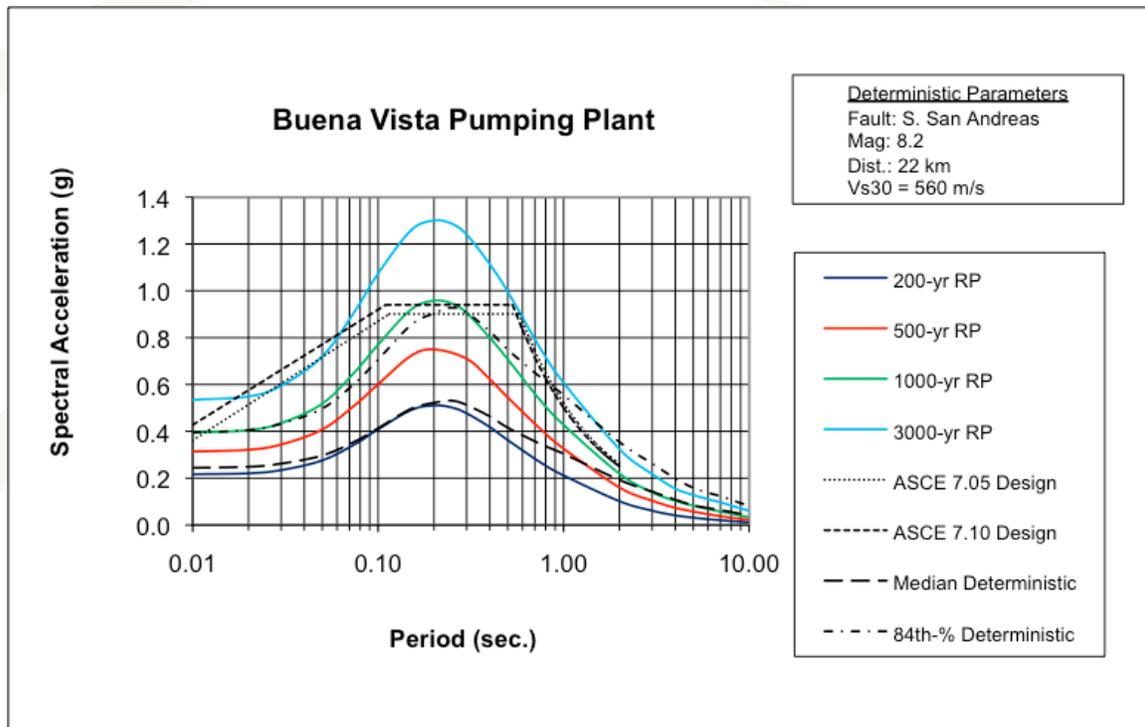


Figure A.15
 Buena Vista Pumping Plant Estimated Ground Motions

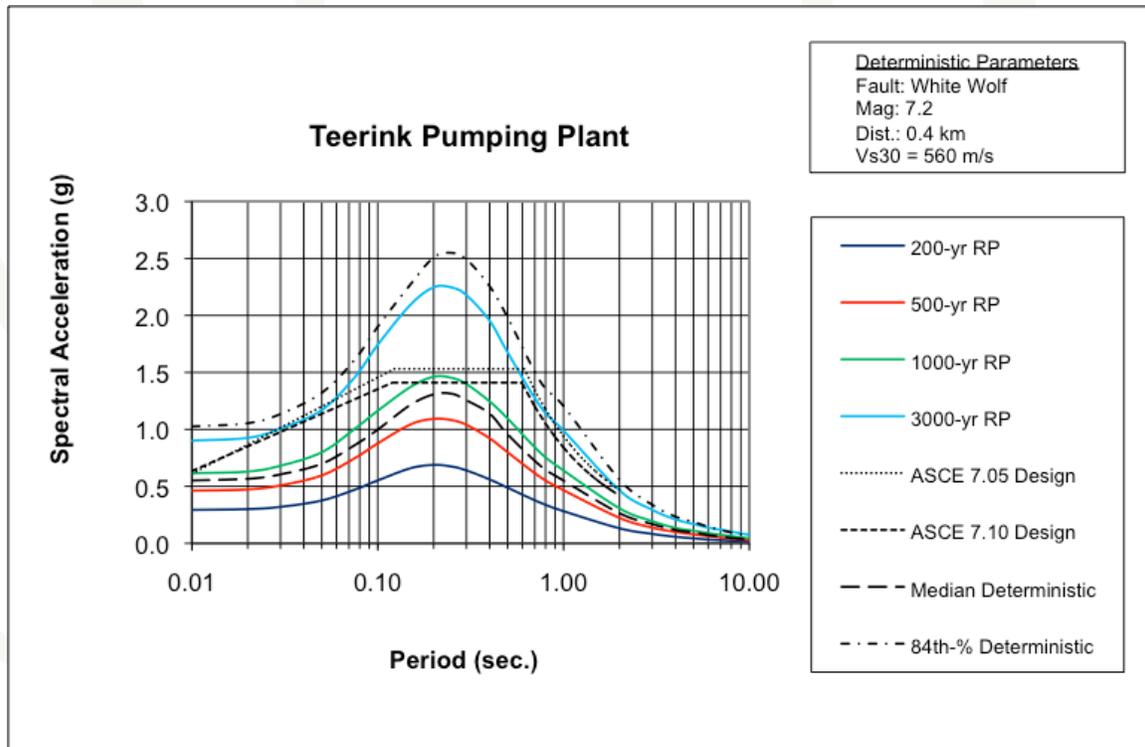


Figure A.16
 Teerink Pumping Plant Estimated Ground Motions

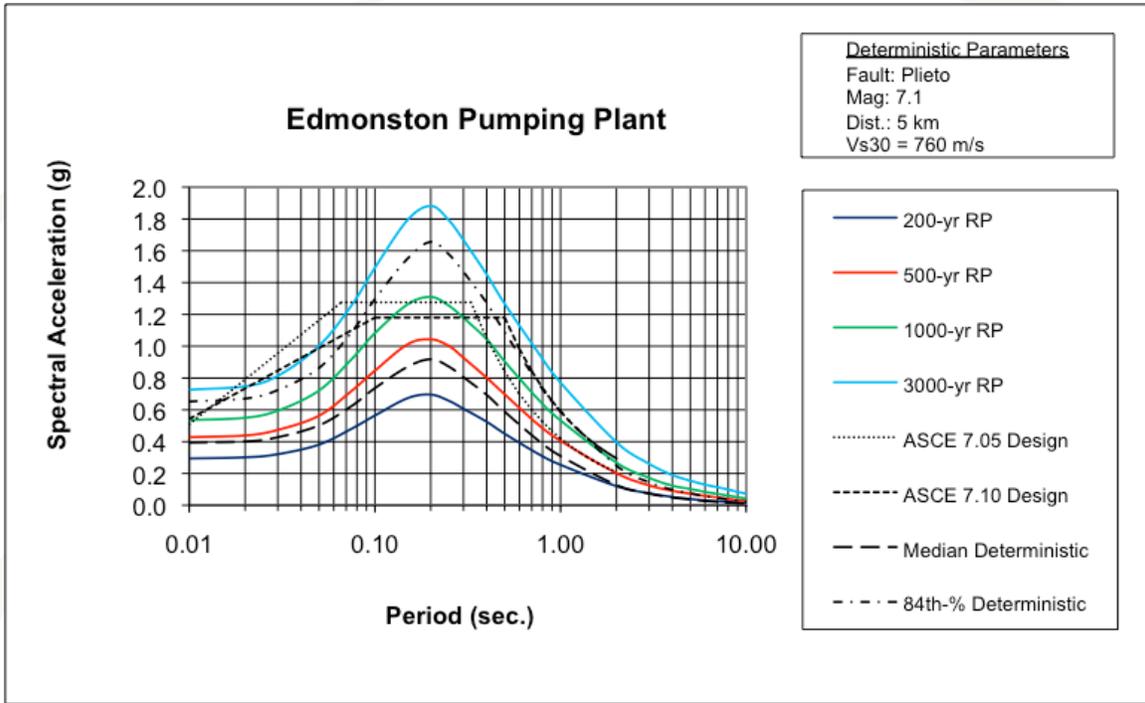


Figure A.17
 Edmonston Pumping Plant Estimated Ground Motions

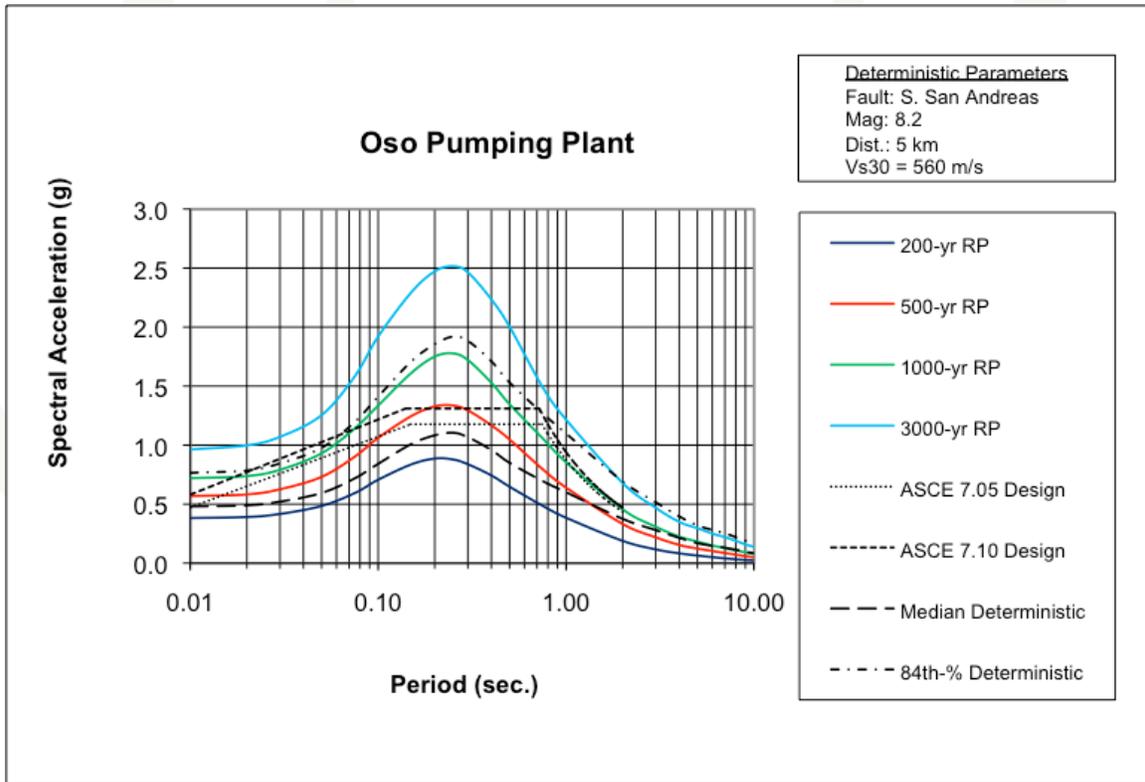


Figure A.18
 Oso Pumping Plant Estimated Ground Motions

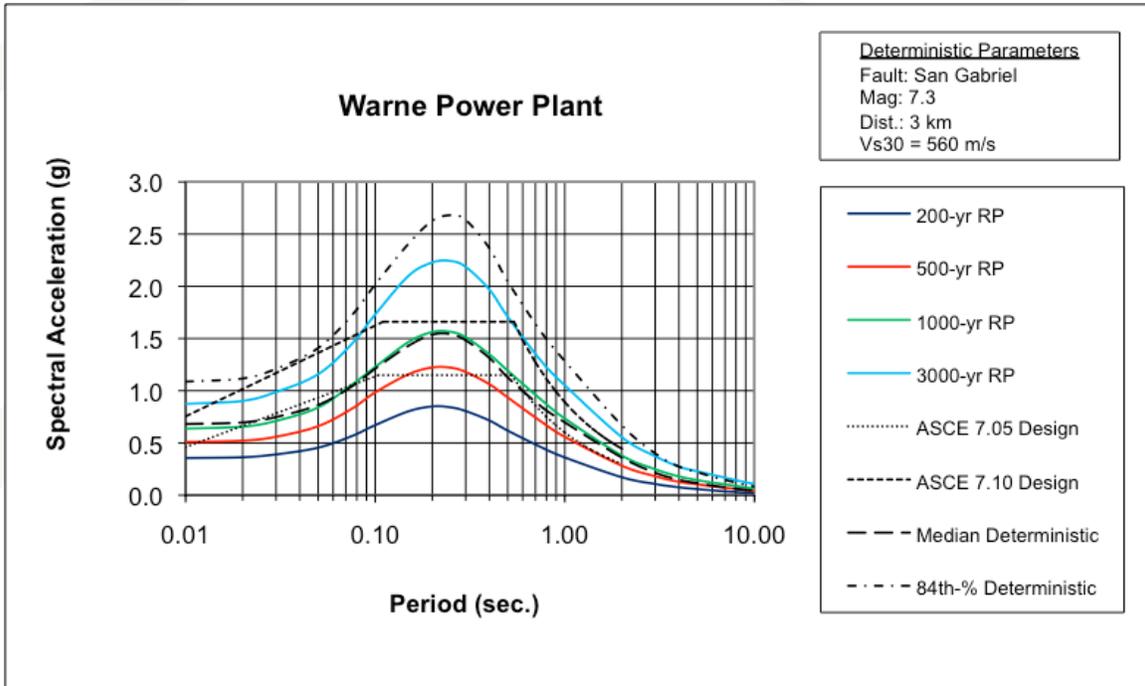


Figure A.19
 Warne Power Pumping Plant Estimated Ground Motions

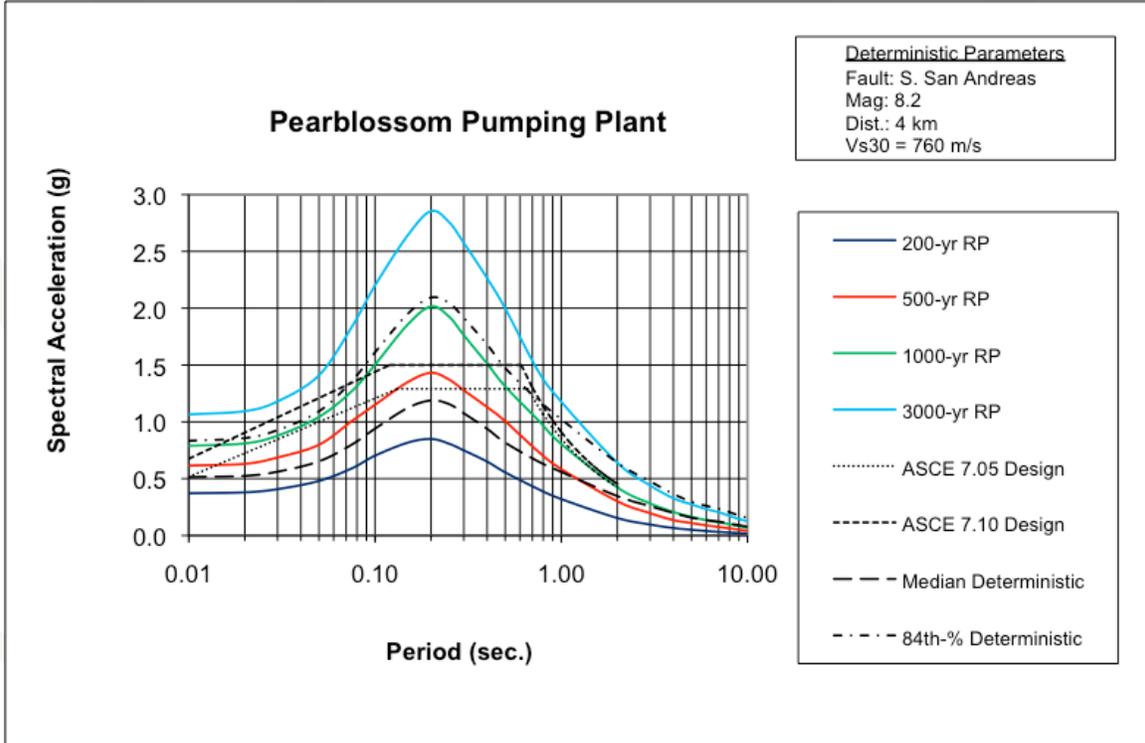


Figure A.20
 Pearblossom Pumping Plant Estimated Ground Motions

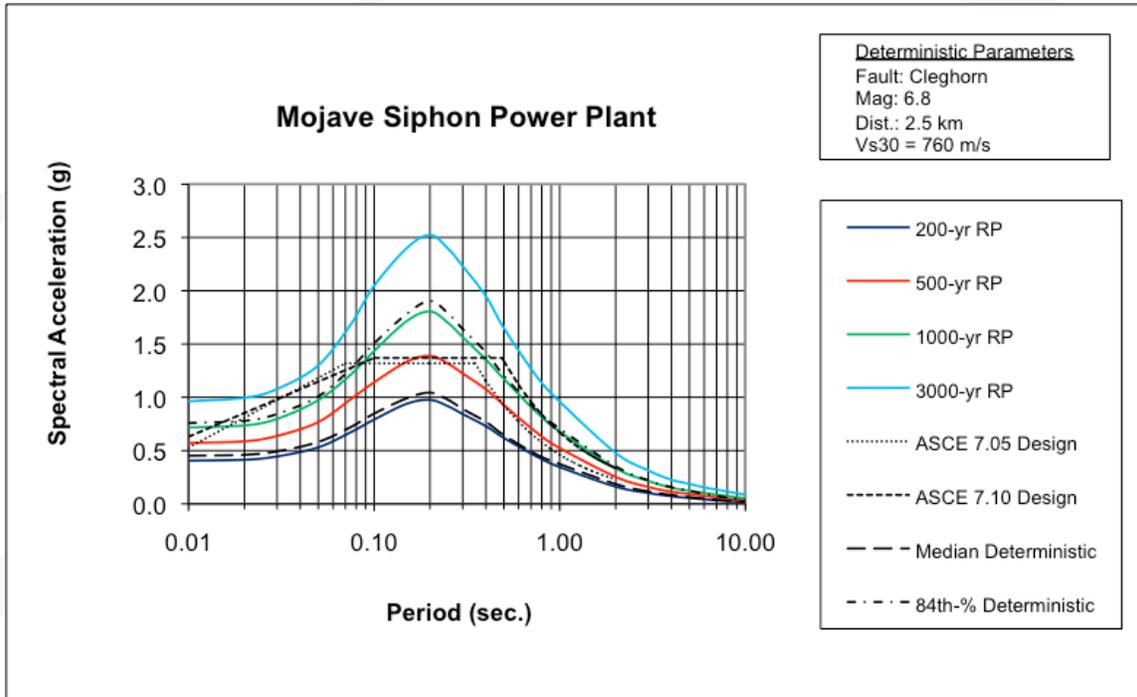


Figure A.21
 Mojave Siphon Power Plant Estimated Ground Motions

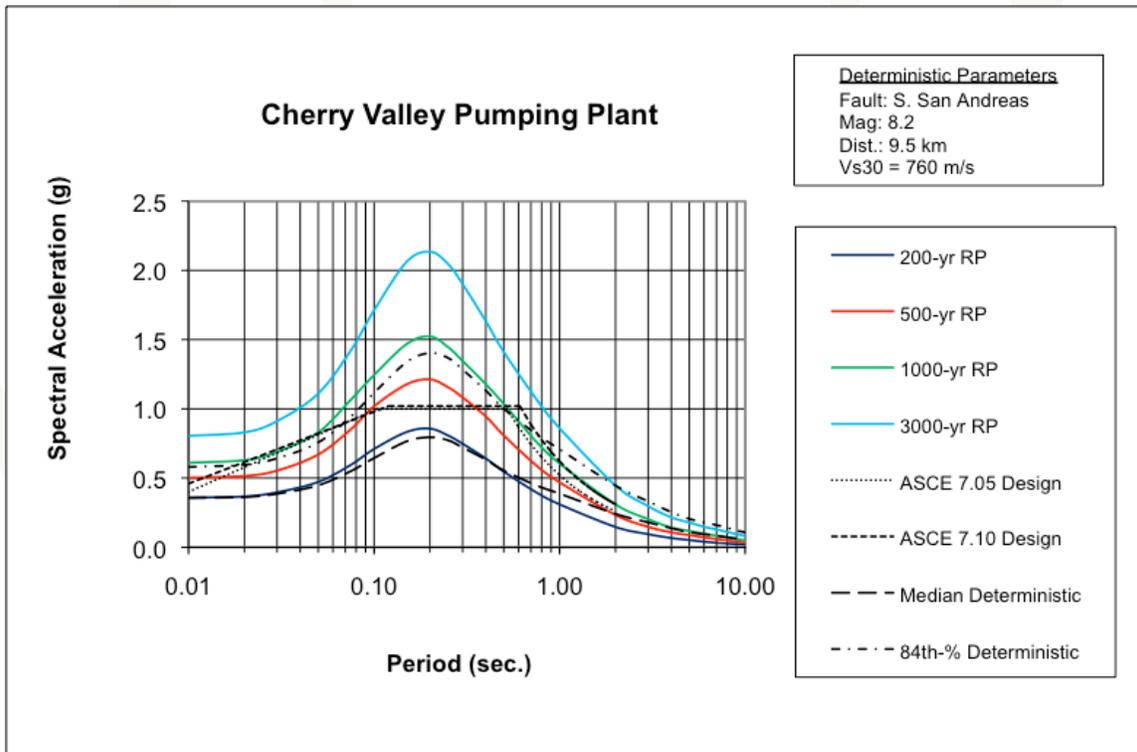
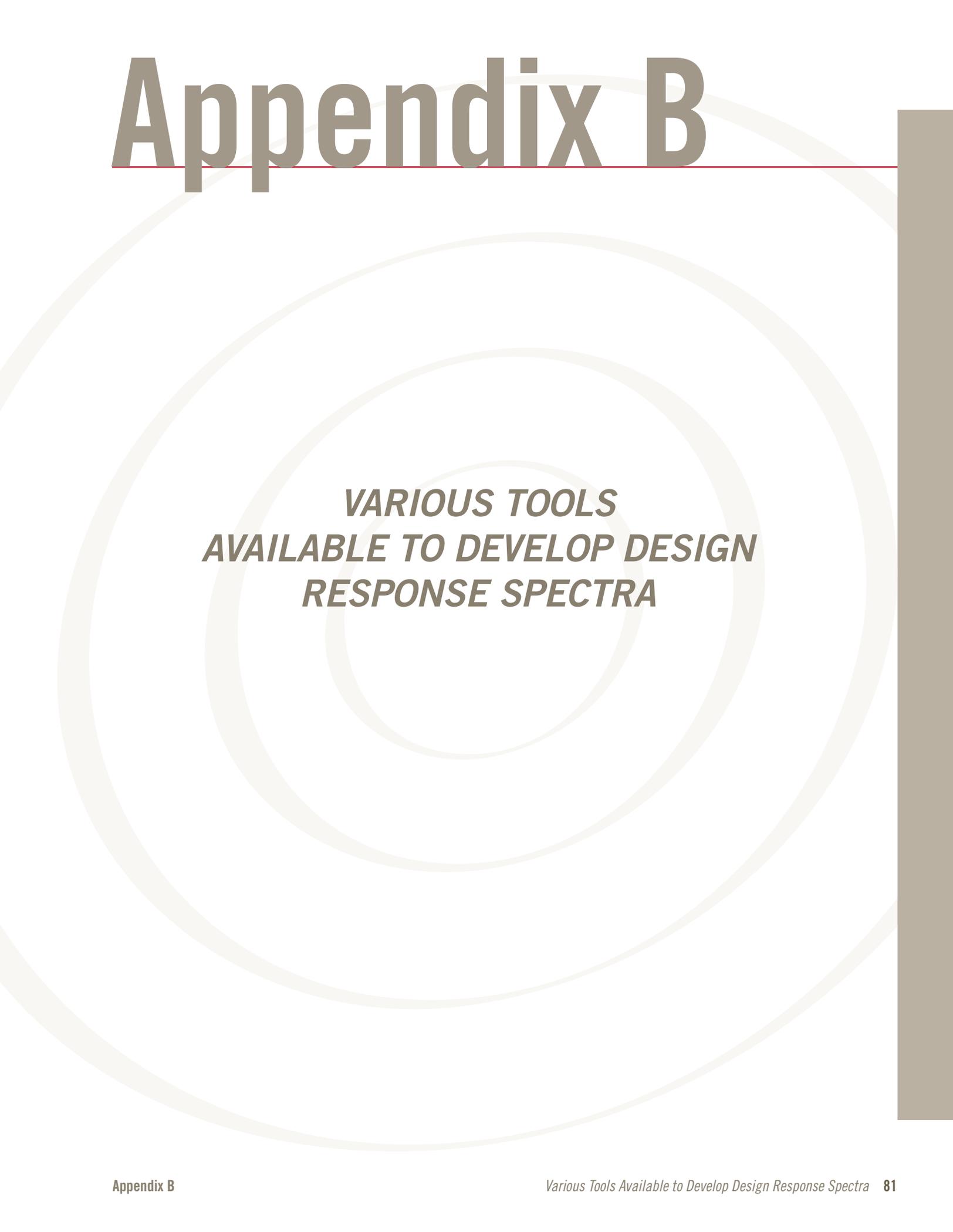


Figure A.22
 Cherry Valley Pumping Plant Estimated Ground Motions

Appendix B



***VARIOUS TOOLS
AVAILABLE TO DEVELOP DESIGN
RESPONSE SPECTRA***

List of available design tools are listed below for use in *preliminary* and final specification of the design spectrum:

- Deterministic PGA map. (http://dap3.dot.ca.gov/shake_stable/references/Deterministic_PGA_Map_8-12-09.pdf). Preliminary spectral curves for several magnitudes and soil classes (Appendix B, Figures B.13-B.27).
- Spreadsheet with preliminary spectral curve data, (http://dap3.dot.ca.gov/shake_stable/references/Preliminary_Spectral_Curves_Data_073009.xls).
- Recommended fault parameters for California faults meeting criteria specified in Appendix B, (http://dap3.dot.ca.gov/shake_stable/references/2007_Fault_Database_120309.xls).
- Deterministic Response Spectrum spreadsheet, (http://dap3.dot.ca.gov/shake_stable/references/Deterministic_Response_Spectrum_072809.xls).
- Probabilistic Response Spectrum spreadsheet, (http://dap3.dot.ca.gov/shake_stable/references/Probabilistic_Response_Spectrum_080409.xls).
- Caltrans ARS Online (Caltrans intranet): http://10.160.173.178/shake2/shake_index2.php, internet: http://dap3.dot.ca.gov/shake_stable/).
- USGS Earthquake Hazards Program website, (<http://earthquake.usgs.gov/research/hazmaps/index.php>).